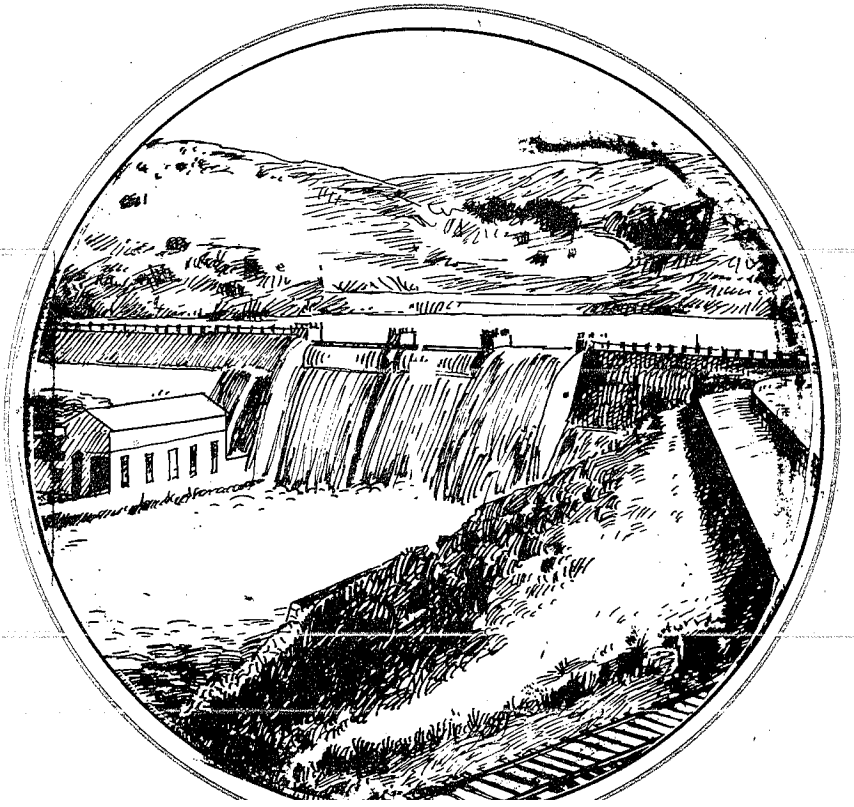
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SANTOSH KUMAR GARG



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IRRIGATION ENGINEERING AND HYDRAULIC STRUCTURES

For

[Civil Engineering degree students ; AMIE (Section B) Exams-New Scheme ; U.P.S.C. and Other State Service Competitions ; and For Professionals]

(Containing updated Questions of Civil Services and Engineering Services Competitions)

By

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DEDICATION

This book is dedicated to the memory of those unknown workers who have lost their lives in the execution of world's gigantic water resources projects.

FOREWORD

I feel both pleasure and pride in being called upon to write the Foreword to this book. This is Shri Garg's second book and he has fully justified the promise shown by his first one—"Water Resources and Hydrology". This is not surprising as he has had a brilliant academic career culminating in **First Class First** Degree in Civil Engineering from Delhi University.

Shri Garg has fully covered the subject of irrigation engineering and the design of hydraulic structures connected with irrigation. The book is expected to be specially useful for students of engineering colleges, and those preparing for the A.M.I.E. Examinations in their spare time. It will also be useful for practising engineers concerned with irrigation engineering. The language has been kept simple, making the book extremely readable.

I wish the author all success in this as well as in future efforts in this direction.

Pritam Singh
Formerly, Chairman,
Central Water Commission,
Govt. of India, New Delhi.

New Delhi

PREFACE TO THE NINETEENTH EDITION

The first edition of this popular book was published in August 1976. At that time, the book contained 25 Chapters with 963 standard sized pages, as compared with the 33 chapters and 1572 large sized pages in this 19th edition. Never, even in his dreams, the author thought at that time, that this book will become the 'darling' of the students and professionals dealing in irrigation and water resources subjects, and shall be followed by students from almost every engineering college of the country, including the prestigious IIT's, D.C.E., Regional Engineering and other Colleges.

The growing popularity of the book continued to push up its sales, encouraging the author to go on revising and enlarging the text, from edition after edition. So far, 18 editions of this book have been sold out with 1,06,600 printed copies, and the present edition is its 19th one, whose 4400 copies are being allowed to be printed.

The continuous revision, edition after edition, had infact caused the book to become too bulky to be handled with convenience. There were suggestions for bifurcating the book into two volumes ; but again there were suggestions for not doing so. The Publishers, therefore, decided to increase the size of the pages used in the book in order to reduce their numbers. This change did help in reducing the thickness of this popular book from about 1900 pages to about 1500 pages.

The M.K.S. units earlier adopted in the book have also been changed to the **S.I. Units,** since these modern international units have by now, gained enough popularity in our country. Various other changes have also been affected at several places in the text, as to make some of the explanations easier and more definite. The question banks of the *Engineering Services* as well as of the *Civil Services* Competitions have also been updated, up to the year 2004. Similar exercise has been affected in respect of the Objective Questions of the AMIE Examinations.

Modern equations developed for the design of irrigation channels in India on the basis of Kennedy's and Kutter's equations have been given, which will help in obtaining more definite and quick solutions for the optimal designs of alluvial canals, besides developing computer programming for their designs. Articles on estimating sediment load likely to enter a reservoir, and reservoir sedimentation studies on existing reservoirs have also been included. In the present 19th edition, thorough revision has been carried out in the first 8 chapters of the book to provide simpler & easier explanations, besides updating the various datas, as to make the book more & more useful to the students, teachers & professionals.

Acknowledgements. In going through the nineteen editions of the book, the author has accumulated an enormous amount of intellectual debts from students and teachers over the years, who had written to him about their satisfaction with the book and extended suggestions for the improvement of its various editions. If the book today, is so strikingly popular, they all deserve much of the credit. The author certainly wants them to know of his gratitudes and openness to further suggestions, and is hence enlisting their names below :

Sh. R. Palnivelu of Government College Coimbatore ; *Sh. M.P. Raghavan* of Government College Salem ; *Sh R. Chandrasekharan* and *Dr. A. Achutyanan* of Regional Engineer-

ing College Calicut ; *Sh. P. Sen,* the then Chief Engineer in Central Water Commission ; *Sh. Punya Prasad Khanal,* an old student of Jaipur Engineering College ; *Sh. V.K. Raju,* Asstt engineer in I & P.D. Deptt. Govt of Andhra Pradesh ; *Mr K. Chandrasekharan Nair,* Asstt Professor of Regional Engineering College Calicut ; *Sh. Mohammad S. Mian,* Head of Water Engg. Natural Resources Development College Lusaka ; *Sh. Biswaroop Ghosh,* an Ex. Student of Bengal Engineering College Calcutta ; *Sh. Soumenda Ghosh,* a student of Burla Engineering College, Sambalpur (Orissa) ; *Sh. K.R. Ganesh Hegde,* Sh. L.V.N. *Malleswara Rao,* Lecturer in College of Engineering Visakhapatnam (Andhra Pradesh) ; *Mr A.L. Lalani,* Lecturer in L.K. College Morbi (Gujarat) ; *Mr. Sasank Sekhar Hota* — Lecturer in Barapada School of Engineering and Technology Balasore (Orissa) ; *Mr. STATES KABASIA* from Nairobi (Kenya) ; *Mr. Shri Ram,* Lecturer in Madan Mohan Malviya Engineering College, Gorakhpur (UP) ; *Mr. Girma G. Hawariat from Harar Ethopia ; Mr. K.R. Thamhane, Mr. Liladhar Roratkar* and *Mr. R.S. Mirje* all students from Govt. College of Engineering Shivaji Nagar (Pune) ; *Professor Arun Kumar* from Delhi College of Engineering ; *Mr. Bishoke Sil* of Irrigation and Waterways Directorate of Govt of West Bengal ; Sh. Vijay Singh, a final year student of H.B.T.I., Kanpur ; Sh. Avadh Pratap Singh, a final year student of MITS Gwalior, etc.

The author further wants to record his highest degree of gratitudes to **Prof. M.D. Pol** of Aurangabad for sending & suggesting modifications at several places in the text, based on which, large scale changes have been made in this 19th edition.

The author also wish to record his heartiest thanks and gratitudes towards **Rajeshwari,** his beloved wife, without whose untiring efforts and tolerance, he would never had been able to write and publish even the first edition of this book at the young age of 30 years.

The author also takes this opportunity to express his reverred gratitudes towards **Prof. Dr. R. Narayanan,** the then Head of the Civil Engineering Department at Delhi College of Engg. (now emirtus Professor at IIT, Chennai), who encouraged him to take up writing work, when he was highly depressed in life, at its prime stage.

New Delhi
August 11, 2005

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Introduction to the Subject

Water is the greatest resource of humanity. It not only helps in survival but also helps in making life comfortable and luxurious. Besides various other uses of water, the largest use of water in the world is made for irrigating lands. *Irrigation, infact, is nothing but "a continuous and a reliable water supply to the different crops in accordance with their different needs".* When sufficient and timely water does not become available to the crops, the crops fade away, resulting in lesser crop yield, consequently creating famines and disasters. Irrigation can, thus, save us from such disasters.

The fact that the provision of irrigation facilities can enhance the yield of our crops by a large extent, can be found from the fact that in Madhya Pradesh State, the crops yield is only 40% more than that in Punjab State, while the cropped area in Madhya Pradesh is about 3 times that in Punjab. The reason is that the gross area under irrigation in Madhya Pradesh is only about 30% as compared to about 91% in Punjab. On an average, in India, the yield from irrigated land is about 2.5 tonnes/hectare ; while that from unirrigated land is about 0.5 tonne/hectare.

It can, therefore, be concluded that if full irrigation facilities are not developed, the production of food grains shall be reduced, as the yield of different crops will be reduced. And if sufficient food grains are not available, the people will remain hungry, leading to all round chaoes, looting and economical destruction of the country, hampering its progress & prosperity. In the light of these facts, it can be easily emphasised that 'irrigation' is inevitable, at least in every tropical* or subtropical* country like India.

The adoption of irrigation practice in our country is not a new thing, as it appears to be, because sufficient proofs are available in Indian history, which confirm that irrigation was being practised not only during the periods of Mughals and Aryans, but even during the periods of Pandavas (about 3150 B.C.). Besides the various ancient books which confirm the above facts, there are ruins of various ancient irrigation works, a few of which are till today existing. For example, the most famous old irrigation work, which is functioning even today, is the **'Grand Anicut'** which was built by Chola rulers in the first century A.C. on the Cauvery river. At the site of the Anicut, the river divides into two branches ; the right branch, which is a lower one, is called *Coleroon*. The Grand Anicut must have been constructed to prevent the flow of water into this low levelled branch (Coleroon) to ensure supplies into Cauvery river, so as to irrigate the fertile Tanjavoor delta land. The importance of irrigation was well recongnised by the Mughal rulers, and as such, *the Western Yamuna Canal* which was built by Ferozshah Tughlaq in the year 1355, was got renovated by Emperor Akbar for irrigating lands in the Hissar district (now in Haryana State) in the year 1568. *The Eastern Yamuna Canal*, was also built by Emperor Mohammad Shah Abdali (1712–1748).

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*__Tropical climate__ is hot all the year with much rain. __Sub-tropical climate__ is the one in which the heat is under control with less hot summers and less cool winters. __Temperate climate__ is represented by warm summers, cool winters, with rain falling evenly throughout the year, or there may be rainy and dry seasons.

During the British regime, starting from the year 1818, considerable attention was paid towads utilising the surface water for irrigation, so as to overcome the frequent famines that used to occur in our country. The first public work of Britishers was the re-renovation of the historic Western Yamuna Canal, which used to provide water to Delhi and the neighbouring areas, but had fallen into disuse during the long period of turmoil. This activity was ofcourse undertaken from considerations of demonstrating the new rule of the *East India Company,* as the emerging rulers of India. It was very much appreciated by the people, and more than that, it was found to be financially very remunerative, yielding a return of 14% in 1847. The Eastern Yamuna Canal built by Emperor Mohammad Shah Abdali, was also soon repaired and renovated in the year 1830.

The 1040 km long *Upper Ganga Canal System*, the longest in the world, was also planned, executed and completed in 1854 for carrying a discharge of 191 cumecs to irrigate 0.62 Mha of land, at a cost of Rs. 2.15 million, *about half of the cost of construction of Taj Mahal.* It is still one of the longest canal systems in the world. Modern engineering in India infact, started from that date, when the first Diploma engineering college was also started at Roorkee in 1846, to provide supporting junior staff for the construction of upper Ganga Canal system. Considerable development of irrigation subsequently took place in the Indus basin* also.

Besides the construction of several irrigation canal projects, such as the *Upper Ganga Canal, the Sirhind Canal, the Lower Chenab Canal* in the Ganga and Indus basins, a lot of work was also done in South India on various rivers, such as *Cauvery, Godavari* and *Krishna.* Construction was taken up and completed on *Khadakvasla dam and its link canals,* the *Periyar dam and its link canals,* etc. It may, however, be stated that the primary emphasis during British regime was on developing extensive irrigation through surface water canals. (The storage irrigation was not executed to any appreciable extent). All the low season flows after monsoon floods were diverted for irrigation for winter (Ravi) season crops, and not to contribute to high yields. The diversion arrangements, when first developed, were also temporary, which used to wash away during the next monsoon floods, and had to be constructed again every year. The **extensive irrigation** (covering a large command area though irrigating only a part of it) was a preferred strategy to overcome the frequent famines over large commands, besides preventing water-logging, since drainage was not developed to keep the costs down. Thus, only about 30% of the canal command was irrigated. The process ended with the delivery of 1 cusec (0.0283 m^3/s) of water at the farm head, and further use and management of water was left to the farmers. There was little agriculture extension, and only three waterings were scheduled. *As spring season approached, river flows dwindled and since there was no upstream storage, the supplies became inadequate. The irrigation for summer (Kharif) period agriculture was not developed. Very few storage and virtually no ground water development was undertaken. There was little hydro-electric development except for some run-of-canal works on Ganga Canal, that too developed just prior to independence. There was no concern for development of drinking water and sanitation facilities, much less flood mitigation. Environmental concerns were totally neglected, as almost all the lean season flows were diverted for irrigation, and the rivers in downstream were left almost bone dry. In the downstream reaches, even the regenerated ground water flows were not allowed to flow freely, and were again diverted for irrigation, again leaving the rivers dry.* But then, it was perhaps

*The *Indus river* having five famous tributaries—the *Satluj,* the *Beas*, the *Ravi*, the *Chenab,* and the *Jhelum* do constitute the Indus basin.

the need of the hour, as it was necessary to stabilise agriculture and mitigate famines. The large scale efforts made by the Britishers resulted in providing irrigation to 28.2 Mha of area, out of total net cultivated area of 116.8 Mha, thus bringing about 24% of the net cultivated area under irrigation. In order to develop irrigation in the country, the first irrigation commission* was even appointed in 1901, which submitted its report in 1903, based on which, large scale canal development works were undertaken, besides preparing blue prints of various important storage projects, including that of the Bhakra dam. Although in undivided India, about 24% of the countrys cropped area came under irrigation, yet with the unfortunate partition of the country, 31% of the country's irrigated area stood transferred to Pakistan, as against the transfer of only 18% of its population.

The independent India in the year 1947 was, thus, left high and dry with only 19% of its cropped area under irrigation ; whereas Pakistan at the time of its creation found 44% of its cropped area under irrigation.

The Indian Government of the independent India, at its very outset, was thus faced to face acute famines and food grain shortages, because most of the country's area was dependent upon natural rainfall, which proved highly unreliable and erratic. Large scale import of wheat from USA as soft term asistance under the famous PL-480 programme, did help the nation to overcome hunger, death and destruction at that time.

Large scale efforts were then made by the Indian Government under the prime ministership of our beloved leader Pt. Jawahar Lal Nehru, to develop and harness our vast water resources, so as to ensure collection of water during monsoons, and its subsequent use for irrigation during non-monsoon periods. **Several dam reservoirs**** were, therefore, planned and constructed across various rivers to store water during rainy season to reduce the fury of floods, and long **canal networks** constructed to move down the stored water during dry weather to the fields, to ensure irrigation supplies to the crops. Hydro-power development was also undertaken as a side product.

The zeal shown in the construction of dams can be gauged by the mere fact that Pt. Jawahar Lal Nehru, the then Prime Minister of India, while inaugurating and dedicating the **Bhakra Dam** to the Nation on 22.10.1963, called it a "*New temple of resurgent India*' and '*Symbol of India's progress*'. The great Pt. Nehru gave a great impetus to the construction of such new temples, and did all what he could, for the execution of several multi-purpose projects, by investing huge funds at a time when money was badly required for other important sectors, like Eduction, Housing, Roads and Railways, Hospitals, etc.

With the construction of multipurpose dams, hydropower became available, which helped to develop tubewell irrigation in the country. This was an important change that took place from early 1960's, when rapid development of tube-wells took place. So much so that, at present, we are using ground water for irrigating about 51% of our gross irrigated area, as against irrigating 33% of the area in 1965-66. Whereas, although development of

*The second irrigation commission was appointed in 1969, and it submitted its report in March 1972.

From less than 300 large dams existing at the beginning of planned development, the number of large dams constructed (excluding about 700 dams under construction) has gone upto about **3600, as to create **live storage capacity of 213 b.cum** on different rivers by the completed projects, and an additional 76 bcum shall be created by the ongoing projects. Projects to further create 108 bcum of live storage are also under consideration of the Govt. of India. Out of such a large number of dam reservoirs, CWC-GOI regularly monitors the storage position of 71 major reservoirs, which together account for a design storage of 113 bcum.

minor irrigation through the use of ground water did help in controlling water–logging in areas where canal irrigation did exist, yet it has resulted in abnormal fall in the ground water tables in several areas, where no attention has been paid to the ground water recharge. The lowered water-tables over vast areas of the country has resulted in making the available ground waters saline and contaminated with fluorides and arsenic, making them unfit for drinking and other domestic requirements*, for which they have largely been used, since olden days**.

Inspite of all such shortcomings and drawbacks, impressive achievements in developing and harnessing our water resources have been made after independence to create an irrigation potential of about 105 Mha (by end of 9th plan ; *i.e.*, March 2002), by incurring an expenditure of about Rs. 1,15,000 crores.

The plan wise investments made under irrigation section during 1951–2002 are reflected in table I.

Table I. Planwise Expenditure Incurred on Irrigation in India

S.No.	*Plan*	*Period*	*Expenditure Incurred (Rs. Crores) in*		
			Major and Medium Irr. Projects	*Minor Irr. Projects**	*Total*
1.	First Plan	1951–56	376	67	443
2.	Second Plan	1956–61	380	162	542
3.	Third Plan	1961–66	576	443	1,019
4.	Annual Plan	1966–69	430	561	991
5.	Fourth Plan	1969–74	1,242	1,173	2,415
6.	Fifth Plan	1974–78	2,516	1,410	3,926
7.	Annual Plan	1978–80	2,079	982	3,061
8.	Sixth Plan	1980–85	7,369	3,417	10,786
9.	Seventh Plan	1985–90	11,107	6,280	17,387
					41,000 (app.)
10.	Two annual Plans and 8th Plan	1990–97			27,000
11.	9th Plan	1997–02			47,000 crores
	Total upto 2001–02				1,15,000 crores

The planwise irrigation potential developed and utilised up to March 2002 (end of 9th plan) in India are shown in Table II. It can be seen from this table that the created irrigation potential has gone up from **22.61 Mha.** In 1951, to **105.33 Mha** by the end of IXth plan (March 02), against the ultimate irrigation potential of **140 Mha** from conventional inbasin storage techniques (Table III). Out of the total created potential of **105.33 Mha,** we have, however, utilised only **83.91 Mha** during the year 2001–02. This has resulted in irrigating about 45% of our total gross cropped area of 189 Mha, which has helped the

*Pl refer "Environmental Engg. Vol I—Water Supply Engineering" by the same author.

**Ground water still provides about 80% of the domestic water supply in rural areas, and about 50% of urban and industrial water requirements (Planning Commission, 1999).

country to achieve self sufficiency in food grains, the production of which has touched about 210 million tonnes (Mt) as against our present consumption of about 160. Mt for 1050 Million population at an average annual per capita consumption of about 150 kg/year. The actual 210 Mt production of food grains is found to be inconsonance with the theoretical production at our observed production rates of 2.5 t/ha for irrigated land, and 1.0 t/ha for unirrigated (rain-fed) land, for 65% of the cropped area growing food grain crops, as :

84 Mha × 0.65 × 2.5 t/ha + (189 – 84) 0.65 × 1.0 t/ha = **205 Mt.**

Table II. Planwise Position of Irrigation Potential Created and Utilised (Cumulative Figs. in Mha)

Plan	*Potential Created*					*Potential Utilised*				
	Major & Medium**	*Minor**			*Total Major Medium Minor*	*Major & Medium*	*Minor*			*Total Major, Medium & Minor*
		Surface Water	*Ground Water*	*Total*			*Surface*	*Ground*	*Total*	
Preeplan upto 1951	9.71	6.40	6.50	12.90	22.61	9.71	6.40	6.50	12.90	22.61
Ist Plan (51-56)	12.19	6.43	7.63	14.06	26.25	10.99	6.43	7.63	14.06	25.05
II Plan (56-61)	14.33	6.45	8.28	14.73	29.06	13.05	6.45	8.28	14.73	27.78
III Plan (61-66)	16.57	6.48	10.52	17.00	33.57	15.17	6.48	10.52	17.00	32.18
Annual Plans (66–69)	18.10	6.51	12.51	19.02	37.12	16.75	6.51	12.51	19.02	35.77
IV Plan (69–74)	20.70	6.96	16.44	23.40	44.20	18.69	6.69	16.44	23.40	42.09
V Plan (74–78)	24.72	7.50	19.80	27.30	52.02	21.16	7.50	19.80	27.30	48.26
Annual Plans (78–80)	26.61	8.00	22.00	30.00	56.61	22.65	8.00	22.00	30.00	52.65
VI Plan (80–85) (Reapprised)	27.70	9.70	27.82	37.52	65.22	23.57	9.01	26.24	35.25	58.82
VII Plan (85–90)	29.92	10.99	36.62	46.61	76.53	25.47	9.97	33.15	43.12	68.59
Annual Plans (90–92)	30.74	11.46	38.89	50.35	81.09	26.32	10.29	36.25	46.54	72.86
VIII Plan (92–97)	32.95	12.09	50.29	62.48	95.43	28.41	8.20	40.09	48.29	80.37
IX Plan (1997–2002)	37.05	13.67	54.61	68.29	**105.33**	31.01	9.05	43.85	52.90	**83.91**

The impressive development of an irrigation potential of about 105 Mha by end of 9th plan has been achieved by execution of about 1232 major, medium and ERM**, irrigation projects, besides a huge number of minor irrigation projects. In addition to this, by the end of IXth plan, 468 (162 Major, 221 Medium & 85 ERM) projects with balance cost of Rs. 863.77 crores have spilled over to the Xth plan. Moreover, 268 new Major, Medium and ERM projects are proposed to be taken up in Xth plan. In all, an additional irrigation potential of 17.91 Mha is likely to be created during the Xth plan ; and out of this,

* Projects having CCA upto 2000 ha are classified as *minor* irrigation projects. Those having CCA of more than 10,000 ha are classified as *major* projects ; and others between 2000 to 10,000 hectares are *medium* proejcts.

**Extension , Renovation and Modernisation (ERM) projects.

Table III : Statewise Ultimate Irrigation Potential

S.No.	Name of State	From Major and Medium Surface Water Schemes	From Minor Irrigation			Total (Major, Medium and Minor)
			Surface Water	Ground Water	Total Minor	
1.	Andhra Pradesh	5.000	2.300	3.960	6.260	11.260
2.	Arunachal Pradesh	0.00	0.150	0.018	0.168	0.168
3.	Assam	0.970	1.000	0.900	1.900	2.870
4.	Bihar	5.224	1.547	4.117	5.664	10.888
5.	Chhatisgarh	1.147	0.081	0.490	0.571	1.718
6.	Goa	0.062	0.025	0.029	0.054	0.116
7.	Gujarat	3.000	0.347	2.756	3.103	6.103
8.	Haryana	3.000	0.050	1.462	1.512	4.512
9.	Himachal Pradesh	0.050	0.235	0.068	0.303	0.353
10.	Jharkhand	1.277	0.353	0.830	1.183	2.460
11.	Jammu & Kashmir	0.250	0.400	0.708	1.108	1.358
12.	Karnataka	2.500	0.900	2.574	3.474	5.974
13.	Kerala	1.000	0.800	0.879	1.679	2.679
14.	Madhya Pradesh	4.853	2.119	9.242	11.361	16.214
15.	Maharashtra	4.100	1.200	3.652	4.852	8.952
16.	Manipur	0.135	0.100	0.369	0.469	0.604
17.	Meghalaya	0.020	0.085	0.063	0.148	0.168
18.	Mizoram	0.000	0.065	0.005	0.070	0.070
19.	Nagaland	0.010	0.070	0.005	0.075	0.085
20.	Orissa	3.600	1.000	4.203	5.203	8.803
21.	Punjab	3.000	0.050	2.917	2.967	5.967
22.	Rajasthan	2.750	0.600	1.778	2.378	5.128
23.	Sikkim	0.020	0.050	0.000	0.050	0.070
24.	Tamil Nadu	1.500	1.200	2.832	4.032	5.532
25.	Tripura	0.100	0.100	0.081	0.181	0.281
26.	Uttar Pradesh	12.154	1.186	16.295	17.481	29.635
27.	Uttaranchal	0.346	0.014	0.504	0.518	0.864
28.	West Bengal	2.3000	1.300	3.318	4.618	6.918
29.	Total UTs	0.098	0.035	0.116	0.151	0.249
	Grand Total (All India)	58.465 ; Say **58.46 Mha**	**17.372**	64.171	81.543 Say **81.54 Mha**	140.008 ; Say **140 Mha**

9.92 Mha is likely to be created through Major and Medium surface irrigation schemes, which are more important than ground water schemes in present days, when ground waters are already over-exploited in various States.

As a matter of fact, a large number of river valley projects have spilled over from plan to plan, mainly because of financial constraints faced by the state Governments. Due to this, despite, huge investments having already been made on thse projects, the country is not able to derive the desired benefits There, in fact, existed 171 Major, 859 Medium and 72 ERM ongoing irrigation projects in the country pendng at various stages of construction by the end of VIIIth plan (*i.e.* end of year 1996–97) with spill over cost of Rs. 75,690 crores. This was a matter a grave concern for the Union Government, and hence measures for expeditious completion of some of the projects which were in advanced stage of completion were initiated by launching an *Accelerated Irrigation Benefits Programme (AIBP)* during 1996-97, under which central financial assistance is being made available to the States by creating special category States (including HP, J & K, Uttranchal, Sikkim and North-eastern States) and General Category States (including all other States except those included in the special category). A *Fast Track Programme* for projects which can be completed in one year has also been started under this scheme wef Feb. 2002. The central financial assistance in this programme include 70% loan and 30% grant for General category States ; and 10% loan and 90% grant for Special category States. For projects which donot come within the preview of Fast Track Programme, an incentive for conversion of loan to grant criteria is provided, if projects are completed on schedule. This extension of central financial assistance to the State Govts. have resulted in disbursement of about Rs. 14,670 crores from 1996-97 to 2003-04, and has resulted in giving a great impetus to the early completion of irrigation projects. All such serious efforts made by the Govt. of India has already resulted in creation of total irrigation potential to *about 105 Mha,* as against the figure of about 95 Mha at the end of 8th plan.

The statewise irrigation potential created and utilised in the country upto the year 2001-02 (*i.e.* as on 31.3.2002) are given in table IV.

The net and gross cropped areas as well as net & gross irrigated areas in various States of India are shown in table V, along with computing annual intensities of irrigation in Col. (9) and plotted in Fig. I. The data used here is of year 1999–2000. The extent of the use of the different sources of water for irrigation is also given in table VI.

Inspite of such large scale efforts made by India, in having generated an irrigation potential of about 105 Mha, we are still, much short of our requirement, since our entire gross cropped area of about 189 M.ha needs irrigation water, which also needs to be considerably increased by producing more than one crop on larger portion of cropped area by increasing cropping intensity to about 150% from the present value of about 134%*.

As can be seen from Col. (4) of Table V, the total cultivated area in the country is about 184 Mha, which is about 60% of the total geographical area of about 329 Mha. The four States of Madhya Pradesh, Maharashtra, Rajasthan and U.P. account for almost 50% of the total cultivated area. For the country as a whole, the gross sown area (189 Mha) exceeds the net sown area (141 Mha) by about 34%.

As a matter of fact, during the last two decades, the net cropped area has not changed much, fluctuating in a narrow range between 140 to 143 Mha ; while the Gross cropped area has increased from 166 Mha to 189 Mha. The cropping intensity has, thus, increased from 118% in 1979 to about 134% in 2002. However, the area under *food grain crops* (like

*Cropping intensity $= \frac{\text{Gross cropped area}}{\text{Net cropped area}} = \frac{189.24 \text{ Mha}}{141.23 \text{ Mha}} = 134\%.$

Table IV : Ultimate, Created and Utilised Statewise Irrigation Potential in India upto End of 9th Plan, i.e. year 2001-02.

S.No.	State	Ultimate Irrigation potential Mha from major, medium & minor surface as well as Ground water	Irrigation potential created (PC) in Mha			Irrigation potential utilised (PU) in Mha		
			Major and medium surface irrigation	Minor irrigation (total) i/e surface Tanks as well as ground water irrigation	total	Major and medium surface irrigation	Minor irrigation (total) surface Tanks as well as ground water irrigation	Total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1.	Andhra Pradesh	11.260	3.303	5.437	8.740	3.052	3.677	6.729
2.	Arunachal Pradesh	0.168	0.000	0.076	0.076	0.000	0.046	0.046
3.	Assam	2.870	0.244	0.890	1.134	0.174	0.587	0.761
4.	Bihar	10.888	2.680	4.743	7.423	1.715	3.074	4.789
5.	Chhatisgarh	1.718	0.923	0.001	0.924	0.761	0.000	0.761
6.	Goa	0.116	0.021	0.021	0.042	0.015	0.019	0.034
7.	Gujarat	6.103	1.430	2.882	4.312	1.301	1.963	3.264
8.	Haryana	4.512	2.100	2.375	4.475	1.850	2.283	4.088
9.	Himachal Pradesh	0.353	0.013	0.223	0.236	0.008	0.188	0.196
10.	Jharkhand	2.460	0.354	0.459	0.823	0.230	0.393	0.623
11.	J & K	1.358	0.180	0.000	0.180	0.169	0.000	0.169
12.	Karnataka	5.974	2.121	1.754	3.875	1.845	1.334	3.179
13.	Kerala	2.679	0.609	0.626	1.235	0.559	0.560	1.119
14.	Madhya Pradesh	16.214	1.387	5.750	7.137	0.876	4.100	4.976
15.	Maharashtra	8.952	3.239	5.262	8.521	2.147	3.553	5.700
16.	Manipur	0.604	0.156	0.049	0.205	0.111	0.029	0.140
17.	Meghalaya	0.168	0.000	0.085	0.085	0.000	0.049	0.049
18.	Mizoram	0.070	0.000	0.013	0.013	0.000	0.011	0.011
19.	Nagaland	0.085	0.000	0.079	0.079	0.000	0.049	0.049
20.	Orissa	8.803	1.827	1.676	2.903	1.794	0.951	2.745
21.	Punjab	5.967	2.543	6.986	9.529	2.486	6.353	8.839
22.	Rajasthan	5.128	2.482	4.839	7.321	2.314	4.295	6.609
23.	Sikkim	0.070	0.000	0.029	0.029	0.000	0.020	0.020
24.	Tamil Nadu	5.532	1.549	3.843	5.392	1.549	3.096	4.645
25.	Tripura	0.281	0.005	0.069	0.074	0.005	0.056	0.061
26.	U.P.	29.635	7.910	16.725	24.635	6.334	13.885	20.219
27.	Uttranchal	0.864	0.280	0.001	0.281	0.185	0.001	0.186
28.	West Bengal	6.918	1.683	3.293	4.976	1.527	2.282	3.809
29.	Total UTs	0.249	0.007	0.096	0.103	0.003	0.090	0.093
	Grand Total (All India)	140.008	37.046	68.282	**105.328**	31.010	52.899	**83.909**

Table V : Statewise Extent of Cultivation & Irrigation done in India (1999-2000)

S.No.	*Name of State*	*Geographical area*	*Total cultivable area i.e. CCA in Mha*	*Net cropped Mha*	*Gross cropped Area Mha*	*Gross irrigated Area Mha*	*%age of gross area under irrign.* $= \frac{G.Irr}{G.Cropped} \times 100$ $= \frac{(7)}{(6)} \times 100$	*Annual intensity of irrigation* $= \frac{Gross\ Irr}{CCA}$ $= \frac{(7)}{(4)} \times 100$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1.	Andhra Pradesh	27.51	15.93	10.610	13.023	5.746	44.12	36.07
2.	Arunachal Pradesh	8.37	0.27	0.166	0.264	0.036	13.64	13.33
3.	Assam	7.84	3.22	2.701	4.093	0.572	13.98	17.76
4.	Bihar	9.42	6.94	5.902	7.919	4.255	53.73	63.31
5.	Chhatisgarh	13.52	5.60	4.135	5.446	0.761	13.97	13.59
6.	Goa	0.38	0.22	0.142	0.171	0.034	19.88	15.45
7.	Gujarat	19.60	12.31	9.667	10.152	3.840	37.83	31.94
8.	Haryana	4.42	3.77	3.552	6.029	5.124	84.99	135.92
9.	Himachal Pradesh	5.57	0.82	0.551	0.957	0.179	18.70	21.82
10.	Jharkhand	7.97	4.19	1.535	2.060	0.553	26.84	13.20
11.	Jammu & Kashmir	22.22	1.05	0.733	1.078	0.438	40.63	41.71
12.	Karnataka	19.18	12.89	10.259	12.097	3.162	26.14	24.53
13.	Kerala	3.89	2.45	2.239	3.002	0.471	15.69	19.22
14.	Madhya Pradesh	30.92	17.23	15.763	20.761	6.330	30.49	36.74
15.	Maharashtra	30.77	21.00	17.691	22.351	3.769	16.86	17.95
16.	Manipur	2.23	0.16	0.140	0.199	0.075	37.69	46.88
17.	Meghalaya	2.24	1.10	0.240	0.266	0.055	20.68	5.00
18.	Mizoram	2.11	0.58	0.091	0.091	0.011	12.09	1.90
19.	Nagaland	1.66	0.64	0.261	0.295	0.073	24.75	11.41
20.	Orissa	15.57	8.09	6.075	8.524	2.512	29.47	31.05
21.	**Punjab**	5.04	4.37	4.238	8.240	7.487	**90.86**	**171.33**
22.	Rajasthan	34.22	25.71	15.509	19.286	6.934	35.95	26.97
23.	Sikkim	0.71	0.11	0.095	0.112	0.016	14.29	14.55
24.	Tamil Nadu	13.01	8.44	5.464	6.519	3.585	54.99	42.48
25.	Tripura	1.25	0.31	0.277	0.420	0.060	14.29	19.35
26.	Uttar Pradesh	24.09	19.43	16.387	25.296	17.515	69.24	90.14
27.	Uttranchal	5.35	1.42	1.198	1.344	0.161	11.98	11.34
28.	West Bengal	8.88	5.93	5.472	9.545	2.491	26.10	42.01
29.	Total UTs	1.09	0.21	0.135	0.189	0.092	48.68	43.81
	Grand Total (All India)	**328.13**	**184.35**	**141.231**	**189.74**	**76.336**	**40.23**	**41.41**

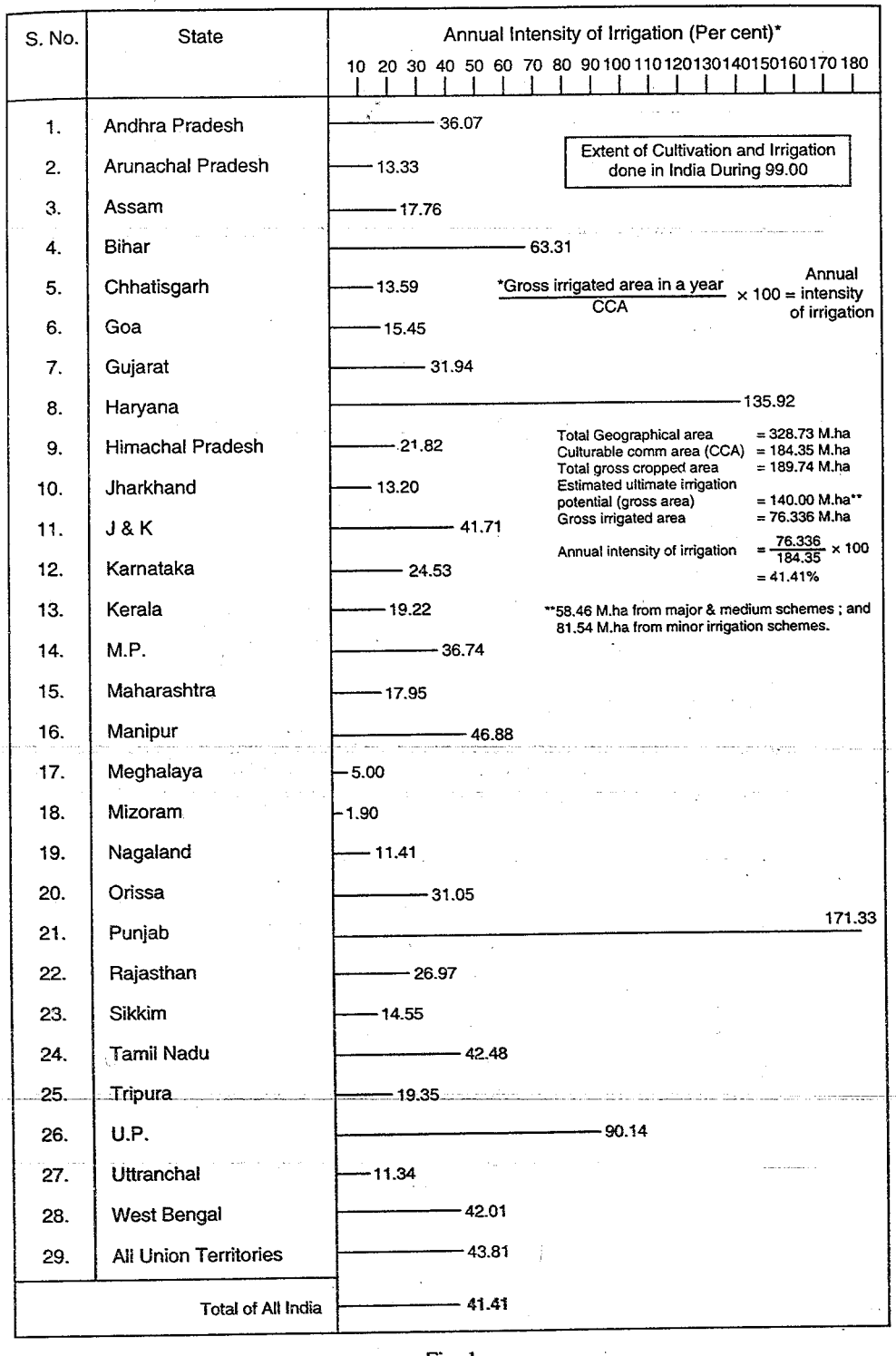

S. No.
State
Annual Intensity of Irrigation (Per cent)*
10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180
1. Andhra Pradesh 36.07
2. Arunachal Pradesh 13.33
3. Assam 17.76
4. Bihar 63.31
5. Chhatisgarh 13.59
6. Goa 15.45
7. Gujarat 31.94
8. Haryana 135.92
9. Himachal Pradesh 21.82
10. Jharkhand 13.20
11. J & K 41.71
12. Karnataka 24.53
13. Kerala 19.22
14. M.P. 36.74
15. Maharashtra 17.95
16. Manipur 46.88
17. Meghalaya 5.00
18. Mizoram 1.90
19. Nagaland 11.41
20. Orissa 31.05
21. Punjab 171.33
22. Rajasthan 26.97
23. Sikkim 14.55
24. Tamil Nadu 42.48
25. Tripura 19.35
26. U.P. 90.14
27. Uttranchal 11.34
28. West Bengal 42.01
29. All Union Territories 43.81
Total of All India 41.41
Extent of Cultivation and Irrigation done in India During 99.00
*Gross irrigated area in a year / CCA × 100 = Annual intensity of irrigation
Total Geographical area = 328.73 M.ha
Culturable comm area (CCA) = 184.35 M.ha
Total gross cropped area = 189.74 M.ha
Estimated ultimate irrigation potential (gross area) = 140.00 M.ha**
Gross irrigated area = 76.336 M.ha
Annual intensity of irrigation = 76.336 / 184.35 × 100 = 41.41%
**58.46 M.ha from major & medium schemes ; and 81.54 M.ha from minor irrigation schemes.

Fig. 1

Table VI. Extent of Irrigation being done from various sources in India

S.No.	*Source*	*%age of Irrigation (app.)*
1.	Canals	39%] surface Irr. (49%)
2.	Tanks and other minor surface sources	15%]
3.	Tubewells and other wells	51% Ground water Irr.
	Total	100%

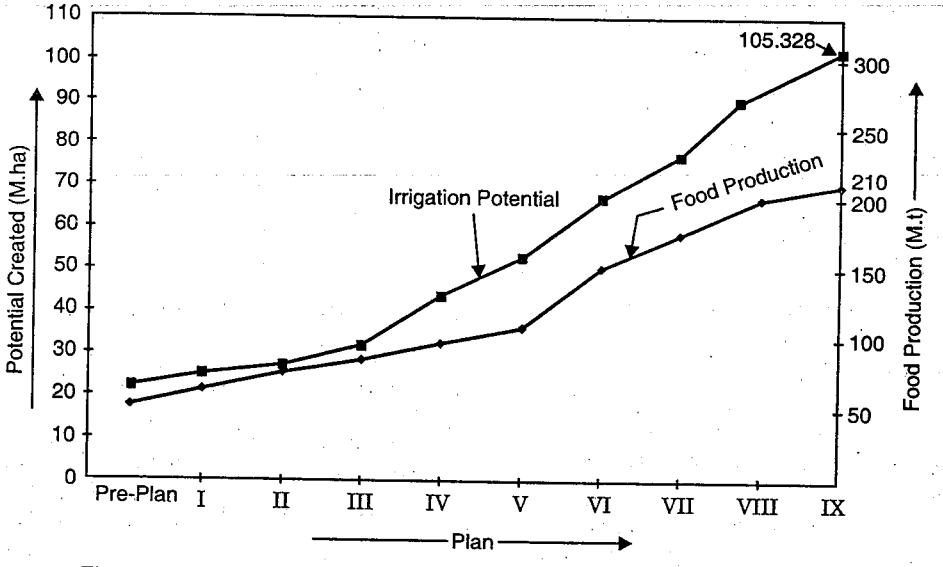


Fig. II. Plan wise growth of Irrigation potential and Food Production in India.

rice, wheat, coarser cereals) has declined from 75% in 1970 to about 65% in 2002. The important aspect of this, however, is the fact that absolute area allotted to **food grain crops** has remained almost unchanged at about **124 Mha** and most of the increase in cropped area has been for the non-food grain crops. Another important feature observed is that the area under wheat and rice has increased from 56 Mha in 1970 to about 69 Mha in 1995 at the expense of coarser cereals. The combined production of rice and wheat during 1970 to 1995 has increased from 66 Mt to 148 Mt, while that of coarse cereals remained constant at about 30 Mt. The increase in production of food grain during the last 32 years (1970–2002) to about 210 tonnes from 108 tonnes (Fig. II) has, of course, largely been caused due to extension of irrigation facilities over a larger cultivated area (about 45%), since the average food grain production per hectare of area for irrigated area has been found to be of the order of 2.5 t/ha, while that for rain-fed area, its value is found to be about 1.0 t/ha. The average food grain production per hectare of cultivated area has, thus, increased from 0.84 t/ha in 1970 to 1.60 t/ha in 2002. It has also been observed that while about 70% of unirrigated area is being used for food grain crops, only about 65% of the irrigated area is being used for food grain crops. With an assumption that 65% of the irrigated as well as unirrigated cultivated area shall be continued to be used for producing food grain crops, the production of food grains in the country based on the already developed irrigation potential of about 105 Mha, as well as on development of the entire ultimate irrigation potential of 140 Mha, are worked out below :

(*i*) Likely food production with utilisation of entire developed irrigation potential of about 105 Mha

= 105 Mha × 0.65 (use factor for food grain crops) × 2.5 t/ha
+ (189 – 105) Mha × 0.65 × 1.0 t/ha
= 171 + 54 = **225 tonnes**

(*ii*) Likely food production with utilisation of entire ultimate irrigation potential of 140 Mha

= 140 Mha × 0.65 × 2.5 t/ha + (189 – 140) Mha × 0.65 × 1.0 t/ha
= 228 + 32 = **260 tonnes**

It can, thus, be seen that even if we are able to create and utilise the entire ultimate irrigation potential of 140 Mha, we can produce only about 260 M.t of food grains, which may just suffice for our future population of the year 2025, which is projected to be about 1350 million (M) at a moderate average food grain per capita consumption level of about 180 kg/year (*i.e.* 0.5 kg/day), although at present, with our population of 1050 M, our average per capita consumption level is hardly 160 kg/year, but the same is likely to go up in future with progressive removal of poverty and better distribution of food grains, amongst the poor rural and urban dwellers.

It can, thus, the concluded that even if we develop the entire ultimate irrigation potential of 140 Mha, which by better management of minor irrigation projects, has already been increased by 27 Mha from its earlier estimated figure of 113 Mha, we can grow food grains, which would be just sufficient to meet our future requirements upto the year 2025, or so.

Efforts will, therefore, have to be made to plan something more for the future, rather than simply targetting to exploit the ultimate irrigation potential of our conventional in basin development methods. One of the proposed method to achieve this aim is to increase the cropping intensity and to promote dry land farming. These measures, however, are not going to create any prominent impact on increasing food grain production. *The only viable method left to meet our future requirement is the* ***interbasin transfer of water from surplus regions to the deficit regions.*** *It may therefore, become imperative to transfer the water from surplus river basins like Brahmaputra, Barak, Mahanadi, Godavari, etc. to the water deficit areas. Interbasin transfer of water,* which is very much required for the country, is not a new concept, since such transfers had been practised even in the 18th and 19th centuries, when canals like *Western Yamuna Canal, Agra Canal, Kurnool-Cuddapah Canal, Periyar-Vaigai Canal,* etc. were constructed to carry waters of river basins of surplus waters over long distances to the regions of deficit waters. *Indira Gandhi Canal (Rajasthan Canal), Ravi-Beas link, Beas-Sutlej link,* and *Sutlej-Yamuna link* (not yet completed due to differences between Punjab and Haryana on sharing of waters) are the modern examples of inter-basin transfer projects of 20th century. **Several additional links*** are still needed to be planned, cleared and got executed to ensure optimum development of country's water resources, which is likely to help in creating an additional irrigation potential of about 35 Mha, besides providing drinking water, generating 34 M kW of hydropower and providing large scale protection against floods.

In addition to developing and harnessing greater water supplies for use by developing inter-basin transfers of water, there also exists an urgent need to **conserve our water supplies,** and to reduce our water demands by developing modern technologies.

*For details of such links, please refer **"International and Interstate River Water Disputes"**, by the same author.

While industrial water demand can be reduced by developing recycling of waste water and by using superior modern methods to reduce consumption of water, the irrigation water demand can be reduced by using faming techniques like **sprinkler irrigation** and **drip irrigation** (called micro irrigation), on a large scale, which utilize the valuable water in an economical manner, by avoiding its wastage through infiltration and evaporation. Realizing its necessity, the Govt. of India has recently started paying greater attention to promote the use of *micro-irrigation*, by ensuring supply of equipment and know-how at subsidized rates to the farmers. A task force to suggest institutional mechanism needed for promoting such technologies and to ensure reaching of the intended benefits to the target group, has also been constituted by the Union Ministry of Finance, Govt. of India. The present status of use of these modern technologies in the country is however, very limited, since only 1.15 Mha (0.7 Mha under sprinkler and 0.45 Mha under drip irrigation) of cropped area is presently estimated to be under such faming techniques, as against 27 Mha found suitable for developing sprinkler and drip-irrigation farming.

It would also be pertinent to mention here that the adoption of intensive irrigation in certain States like Punjab, Orissa, Andhra Pradesh, etc. has caused alarming rise in the ground water-tables, which has resulted in causing water-logging and reduced crop yields. The non-provision of surface and sub-surface drainage and improper management of irrigation command area has compounded the problem to such an extent that a lot of land has become saline and even alkaline, making it unfit for cultivation. The land affected due to these reasons is estimated to be about 6 Mha (2.7 Mha affected by water-logging + 3.06 Mha affected by salinity + 0.24 Mha affected by alkalinity) **Conjunctive use** of surface and ground water collectively together is an accepted solution for removing water-logging problem, to enhance our crop yields. There is a large scope of making such improvements in alluvial plains, particularly in the *Gangetic plain, the coastal areas of Orissa and Anhdra Pradesh, the Brahmaputra Valley, the Cauvery delta*, and in parts of the Narmada basin.

Although irrigation facilities are so very important and essential for the development of any nation, they are complex and intricate. *Proper development of water resources involves the most economical exploitation of the entire available water of a country to meet the needs of its various regions, not only for irrigation but also for drinking, domestic and industrial requirements which are in no way, less important. Such optimum developments of water resources must also give due consideration to* **ecological aspects** *by proper allocation of low river flows for* **environmental conservation,** *rather than diverting them entirely for irrigation or water supply demands.* Aiming for such an **optimum and integrated utilisation** of the country's entire water-resources, makes this field very complex, intricate, and interesting too. It is here, that the real job of an Irrigation engineer or more precisely, a water-resources expert, comes into play.

How to plan our available water sources to fulfil requirements of various sectors, and to ensure maximum benefits keeping in view the socialistic distribution of the achieved benefits within the available funds, is the real tough job. The design of various irrigation works is another job, which must be accomplished with a fair degree of economy and correctness. Here is a book, which provides the basic fundamental principles, broad guide lines, and details of this intricate and interesting field in a simple language. It is hoped that this book will prove useful to every student, to every teacher, as well as to every design and field engineer.

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*Explained in article 1.6(7) and 1.6(6), respectively.

1

Irrigation Techniques and Quality of Irrigation Water

1.1. Definition of Irrigation

Plants are living beings and do require water and air for their survival, as do human beings require. Their requirement of water varies with their type. Different types of plants require different quantities of water, and at different times, till they grow up completely. Water is normally supplied to these plants by nature through direct rain or through the flood waters of rivers which inundate large land areas during floods. The flood water may saturate the land before the flood is subsided. The water absorbed by the land during floods, supplements the water requirement of the crop during dry season. These natural processes, whereby, the water is supplied to the crops for their growth, are dependent upon 'nature' or 'God', whatever we may call it. Sometimes, there may be very heavy rains creating serious floods and damaging the crops, and sometimes, there may not be any rains at all, creating scarcity of water for the crops. Thus, famine and scarcity conditions are created. In his bid to control the nature, man discovered various methods by which the water can be stored during the periods of excess rainfall, and to use that stored water during periods of 'less rainfall' or 'no rainfall'. The art or the science by which it is accomplished, is generally, termed, as irrigation. *Irrigation may, therefore, be defined as the science of artificial application of water to the land, in accordance with the 'crop requirements' throughout the 'crop period'* for full-fledged nourishment of the crops.

1.2. Necessity of Irrigation in India

India is a tropical country with a vast diversity of climate, topography and vegetation. Rainfall in India, varies considerably in its place of occurrence, as well as in its amount. Even at a particular place, the rainfall is highly erratic and irregular, as it occurs only during a few particular months of the year. Crops cannot, therefore, be raised successfully, over the entire land, without providing artificial irrigation of fields.

More than seventy percent of our population, directly depends on agriculture, and the remaining depends indirectly on agriculture. Out of a total geographical area of about 328 million hectares, about 184 million hectares is the cultivable area. In order to save this area from the complete wishes and vagaries of nature, and to ensure full growth of crops, it is necessary to provide adequate artificial irrigation facilities. In order to achieve this, the Indian Government is trying hard and spending enormously to provide irrigation facilities for the entire cultivable land. So far developed irrigation facilities in India have been shown under "Introduction" in the previous pages.

1.3. Advantages of Irrigation

Every irrigation project is designed, keeping in view of its economics, *i.e.* the expenditure likely to be incurred and the benefit likely to occur. There is a capital investment on the project and the future recurring charges for maintenance and operation. The project estimate is generally sanctioned when the benefit gives at least about 8% interest on the capital outlay. Sometimes, unproductive projects are also sanctioned in view of their *general public benefits.*

There is hardly any point in emphasizing the importance and advantages of irrigation during the times of acute food shortages and growing population of our country.

Even then, some of the advantages of irrigation are summarised below:

(1) **Increase in Food Production.** Irrigation helps in increasing crop yields, and hence, to attain self-sufficiency in food.

(2) **Optimum Benefits.** Optimum utilisation of water is made possible by irrigation. By optimum utilisation, we generally mean, obtaining maximum crop yield with required amount of water. In other words, yield will be smaller for any quantity lesser than or in excess of this optimum quantity.

(3) **Elimination of Mixed Cropping.** In the areas, where irrigation is not assured, generally mixed cropping is adopted. *By mixed cropping, we mean, sowing together of two or more crops in the same field.* If the weather conditions are not favourable to one of the crops, they may be better suitable for the other; and thus, the farmer may get at least some yield. Mixed cropping, is thus, found necessary and also economical when irrigation facilities are lacking, and especially during periods of *Crash programmes* in under-developed countries. But if irrigation is assured, mixed cropping can be eliminated.

Mixed cropping is generally not acceptable, because different crops require different types of field preparations and different types of waterings, manurings, etc. If two crops are mixed together, the field preparations, waterings, manurings, etc. cannot be made to suit the special needs of either. Moreover, during the time of harvesting, the crops get intermixed with each other, reducing the purity of each other. But, when regular and permanent water supply is assured, a single superior crop can be sown, depending upon the conditions of the soil and the needs of the country.

(4) **General Prosperity.** Revenue returns with well developed irrigation, are sometimes, quite high, and helps in all round development of the country and prosperity of the entire nation and community.

(5) **Generation of Hydro-electric power.** Cheaper power generation can be obtained from water development projects, primarily designed for irrigation alone. Canal outlets from dams and Canal falls on irrigation canals can be used for power generation. For example, Ganga and Sarda Canals, constructed for irrigation, are now generating hydro-electric power as a side product, up to about 80,000 kilo-watts.

(6) **Domestic Water Supply.** Development of irrigation facilities in an area helps in augmenting the water supply in nearby villages and towns, where other sources of water are not available or are scarcely available. It also helps in providing drinking water for animals, and water for swimming, bathing, etc.

(7) **Facilities of Communications.** Irrigation channels are generally provided with embankments and inspection roads. These inspection paths provide good roadways to the villagers for walking, cycling or sometimes even for motoring.

(8) **Inland Navigation.** Sometimes, larger irrigation canals can be used and developed for navigation purposes.

(9) **Afforestation.** Trees are generally grown along the banks of the channels, which increase the timber wealth of the country and also help in reducing soil erosion and air polluition.

1.4. Disadvantages and Ill-Effects of Irrigation

(1) Irrigation may contribute in various ways to the problem of water pollution. One of these is the seepage into the ground water of the nitrates, that have been applied to the soil as fertilizer. Sometimes, up to 50% of nitrates applied to the soil, sinks into the underground reservoir. The underground water may thus get polluted, and if consumed by people through wells, etc., it is likely to cause diseases such as anemia. Whether it will ultimately affect the fishing on way to the sea, or as the tides carry the polluted water ahead into the ocean, is yet a matter of research.

(2) Irrigation may result in colder and damper climate, resulting in marshy lands and breeding of mosquitoes, causing outbreak of diseases like malaria & dengu.

(3) Over-irrigation may lead to water-logging* and may reduce crop yields.

(4) Procuring and supplying irrigation water is complex and expensive in itself. Sometimes, subsidised cheaper water has to be provided at the cost of the government, which raduces revenue returns.

1.5. Types of Irrigation

Irrigation may broadly be classified into :

1. *Surface irrigation* ; and 2. *Sub-surface irrigation*

(1) **Surface irrigation** can be further classified into :

(*a*) Flow irrigation ; and (*b*) Lift irrigation.

When the water is available at a higher level, and it is supplied to lower level, by the mere action of gravity, then it is called *Flow Irrigation.* But, if the water is lifted up by some mechanical or manual means, such as by pumps, etc. and then supplied for irrigation, then it is called *Lift Irrigation.* Use of wells and tubewells for supplying irrigation water fall under this category of irrigation.

Flow irrigation can be further sub-divided into :

(*i*) Perennial irrigation, and

(*ii*) Flood irrigation.

(*i*) *Perennial Irrigation.* In perennial system of irrigation, constant and continuous water supply is assured to the crops in accordance with the requirements of the crop,

* For detailed description of water-logging, please refer Chapter 6.

throughout the 'crop period'. In this system of irrigation, water is supplied through storage canal head works and canal distribution system.

When irrigation is done from the direct runoff of a river, or by diverting the river runoff into some canal by constructing a diversion weir or a barrage across the river, it is called *Direct Irrigation.* Ganga Canal System is an example of this type of irrigation. But, if a dam is constructed across a river to store water during monsoons, so as to supply water in the off-taking channels during periods of low flow, then it is termed as *Storage Irrigation.* Ram-Ganga Dam project in U.P. is an example of storage type of irrigation system. In the regions of peninsula India, where rivers are generally seasonal, storage irrigation is an absolute necessity, whereas, in Indo-gangetic region, direct irrigation is feasible, since the rivers are perennial, getting their supplies from the melting of snow. Direct irrigation is always simple, easy and economical. The perennial system of irrigation, is most important and is mostly practised in India.

(*ii*) *Flood Irrigation.* This kind of irrigation, is sometimes called as *inundation irrigation.* In this method of irrigation, soil is kept submerged and thoroughly flooded with water, so as to cause thorough saturation of the land. The moisture soaked by the soil, when occasionally supplemented by natural rainfall or minor waterings, brings the crop to maturity.

(2) **Sub-surface Irrigation.** It is termed as sub-surface irrigation, because in this type of irrigation, water does not wet the soil surface. The underground water nourishes the plant roots by capillarity. It may be divided into the following two types :

(*a*) Natural sub-irrigation ; and

(*b*) Artificial sub-irrigation.

(*a*) *Natural sub-irrigation.* Leakage water from channels, etc., goes underground, and during passage through the sub-soil, it may irrigate crops, sown on lower lands, by capillarity. Sometimes, leakage causes the water-table to rise up, which helps in irrigation of crops by capillarity. When underground irrigation is achieved, simply by natural processes, without any additional extra efforts, it is called natural sub-irrigation.

(*b*) *Artificial sub-irrigation.* When a system of open jointed drains is artificially laid below the soil, so as to supply water to the crops by capillarity, then it is known as artificial sub-irrigation. It is a very costly process and hence, adopted in India on a very small scale. It may be recommended only in some special cases with favourable soil conditions and for cash crops of very high return. Sometimes, irrigation water may be intentionally collected in some ditches near the fields, the percolation water may then come up to the roots through capillarity.

1.6. Techniques of Water Distribution in the Farms

There are various ways in which the irrigation water can be applied to the fields. Their main classification is as follows :

(1) Free flooding
(2) Border flooding
(3) Check flooding
(4) Basin flooding
(5) Furrow irrigation method
(6) Sprinkler irrigation method
(7) Drip irrigation method.

These methods are briefly discussed below :

(1) **Free flooding or Ordinary flooding.** In this method, ditches are excavated in the field, and they may be either on the contour or up and down the slope. Water from these ditches, flows across the field. After the water leaves the ditches, no attempt is made to control the flow by means of levees, etc. Since the movement of water is not restricted, it is sometimes called **wild flooding.** Although the initial cost of land preparation is low, labour requirements are usually high and water application efficiency is also low. Wild flooding, is most suitable for close growing crops, pastures, etc., particularly where the land is steep. Contour ditches called laterals or subsidiary ditches, are generally spaced at about 20 to 50 metres apart, depending upon the slope, texture of soil, crops to be grown, etc. This method may be used on rolling land (topography irregular) where borders, checks, basins and furrows are not feasible.

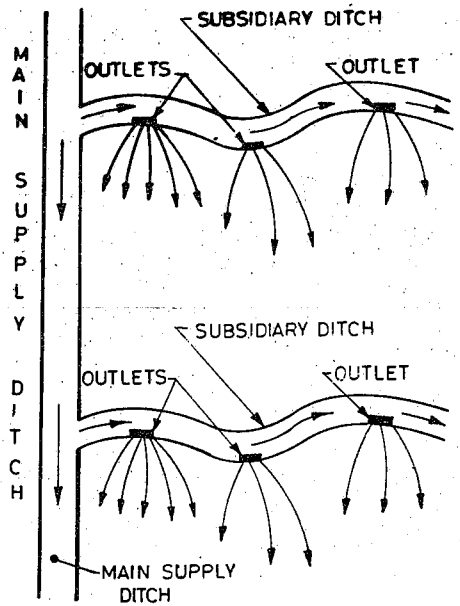


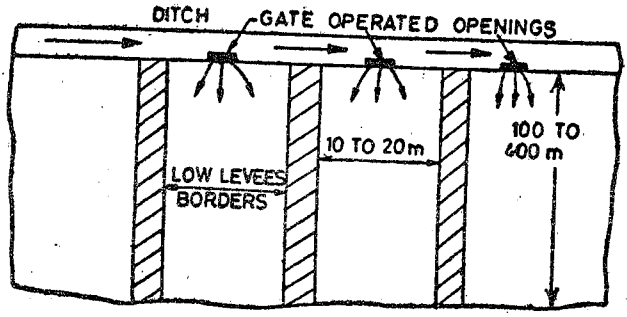
Fig. 1.1. Free flooding (plan view).

(2) **Border flooding.** In this method, the land is divided into a number of strips, separated by low levees called *borders*. The land areas confined in each strip is of the order of 10 to 20 metres in width, and 100 to 400 metres in length, as shown in Fig. 1.2. Ridges between borders should be sufficiently high to prevent overtopping during irrigation.

To prevent water from concentrating on either side of the border, the land should be levelled perpendicular to the flow. Water is made to flow from the supply ditch into each strip. The water flows slowly towards the lower end, and infiltrates into the soil as it advances. When the advancing water reaches the lower end of the strip, the supply of water to the strip is turned off.

The *supply ditch,* also called *irrigation stream,* may either be in the form of an earthen channel or a lined channel or an underground concrete pipe having risers at intervals. The size of the supply ditch depends upon the infiltration rate of the soil, and the width of the border strip. Coarse textured soils with high infiltration rates will require high discharge rate and therefore larger supply ditch, in order to spread water over the entire strip *rapidly,* and to avoid excessive losses due to deep percolation at the upper reaches. On the other hand, fine textured soils with low infiltration rates, require smaller ditches to avoid excessive losses due to run off at the lower reaches.

A relationship between the discharge through the supply ditch (Q), the average depth of water flowing over the strip (y), the rate of infiltration of the soil (f), the area



(*a*) Plan view.



(*b*) Photographic view.

Fig. 1.2. Border flooding.

of the land irrigated (A), and the approximate time required to cover the given area with water (t), is given by the equation :

$$t = 2.3 \cdot \frac{y}{f} \cdot \log_{10}\left(\frac{Q}{Q - fA}\right) \qquad ...(1.1)$$

where Q = Discharge through the supply ditch

y = Depth of water flowing over the border strip

f = Rate of infiltration of soil

A = Area of land strip to be irrigated

t = Time required to cover the given area A.

The above equation can be obtained by considering small area (dA) of the border strip of area A, as shown in Fig. 1.3. Let us also assume that in time dt, water advances

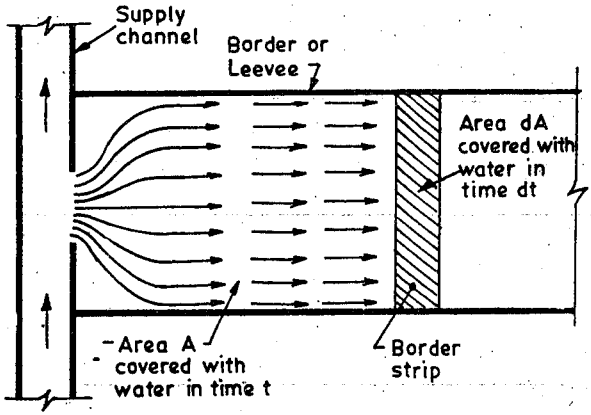


Fig. 1.3.

over this area dA. Now, the volume of water that flows to cover this area would be $y \cdot dA$, because y is the water depth over this area. Also, during the same time dt, the volume of water that percolates into the soil over the area A would be $f \cdot A \cdot dt$. The total quantity of water supplied to the strip during time dt would be $Q \cdot dt$, and also equal to $y \cdot dA + f \cdot A \cdot dt$.

$$\therefore \quad Q \cdot dt = y \cdot dA + f \cdot A \cdot dt \qquad \ldots(i)$$

or
$$dt = \left(\frac{y \cdot dA}{Q - f \cdot A} \right) \qquad \ldots(ii)$$

Considering y, f, and Q as constants, and integrating the above equation, we get

$$t = \frac{y}{f} \log_e \left(\frac{Q}{Q - f \cdot A} \right) + \text{a constant of integration } (K)$$

But at $t = 0$, $A = 0$

$$\therefore \quad K = \frac{y}{f} \log_e 1 = 0 \quad (\because \ \log_e 1 = 0)$$

$$\therefore \quad t = \frac{y}{f} \log_e \left(\frac{Q}{Q - fA} \right)$$

or
$$t = 2.3 \cdot \frac{y}{f} \log_{10} \left(\frac{Q}{Q - fA} \right), \quad \text{which is the above given equation (1).}$$

This equation can be further written as

$$\frac{t \cdot f}{2.3y} = \log_{10} \left(\frac{Q}{Q - fA} \right). \quad \text{Now, let } \frac{t \cdot f}{2.3y} = x$$

Then,
$$x = \log_{10} \left(\frac{Q}{Q - fA} \right)$$

or
$$10^x = \frac{Q}{Q - fA}$$

or $$Q \cdot 10^x - f \cdot A \cdot 10^x = Q$$

or $$Q\,(10^x - 1) = f \cdot A \cdot 10^x$$

or $$A = \frac{Q\,(10^x - 1)}{f \cdot 10^x}$$

Further, considering the maximum value of $\frac{10^x - 1}{10^x} \approx 1$,

we get $$A_{max} = \frac{Q}{f} \qquad ...(1.2)$$

This equation enables us to determine the maximum area that can be irrigated with a supply ditch of discharge Q and soil having infiltration capacity, f.

It can also be inferred from this equation that the discharge per unit area of the border strip (Q/A) should be varied according to the infiltration capacity of the soil (f), otherwise loss of water will take place.

This method of field irrigation is becoming very popular among the cultivators. Shorter and narrower strips are found to be more efficient. Entry of water into the strips is generally controlled by placing a jet in the supply ditch, at the head of each strip.

Example 1.1. *Determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube-well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hr, and the average depth of flow on the field as 10 cm.*

Also determine the maximum area that can be irrigated from this tube well.

Solution. $A = 0.04$ hectares

$$= 0.04 \times 10^4 \text{ m}^2 = 400 \text{ m}^2.$$

$$Q = 0.02 \text{ cumecs} = 0.02 \text{ m}^3/\text{sec} = 0.02 \times 60 \times 60 \text{ m}^3/\text{hr} = 72 \text{ m}^3/\text{hr}.$$

$$f = 5 \text{ cm/hr} = \frac{5}{100} \text{ m/hr} = 0.05 \text{ m/hr}. \quad ; y = 10 \text{ cm} = 0.1 \text{ m}.$$

Using eqn. (1.1), we have

$$t = 2.3 \frac{y}{f} \log_{10}\left(\frac{Q}{Q - fA}\right)$$

or $$t = 2.3 \frac{0.10}{0.05} \log_{10}\left(\frac{72}{72 - 0.05 \times 400}\right) \text{hr}.$$

$$= 2.3 \times 2 \log_{10} (72/52) \text{ hr.} = 0.65 \text{ hr} = 39 \text{ minutes.} \quad \textbf{Ans.}$$

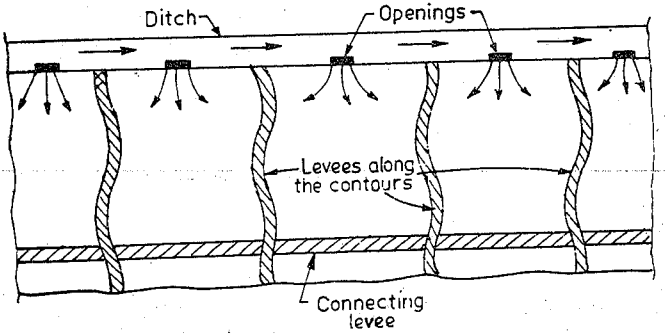
Maximum area that can be irrigated is given by eqn. (1.2) as

$$A_{max} = \frac{Q}{f} = \frac{72}{0.05} \text{ m}^2 = 1440 \text{ m}^2 = \frac{1440}{10^4} \text{ hectares} = 0.144 \text{ hectares.} \quad \textbf{Ans.}$$

Note. After irrigating this much of area, surface flow will stop, and deep percolation will start.

(3) **Check flooding.** Check flooding is similar to ordinary flooding except that the water is controlled by surrounding the *check* area with low and flat levees. Levees are

generally constructed along the contours, having vertical interval of about 5 to 10 cm. These levees are connected with cross-levees at convenient places as shown in Fig. 1.4. The confined plot area varies from 0.2 to 0.8 hectare.



(*a*) Plan view.



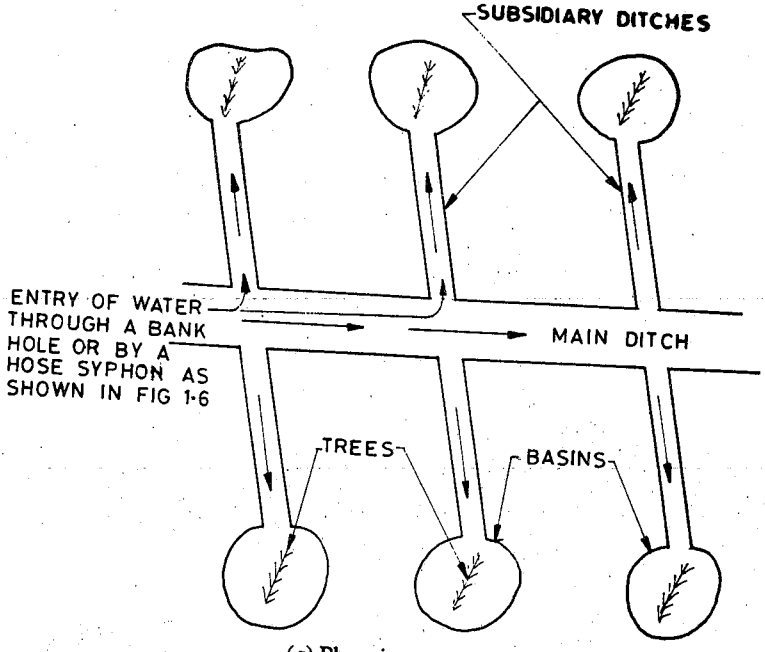
(*b*) Photographic view.

Fig. 1.4. Check flooding.

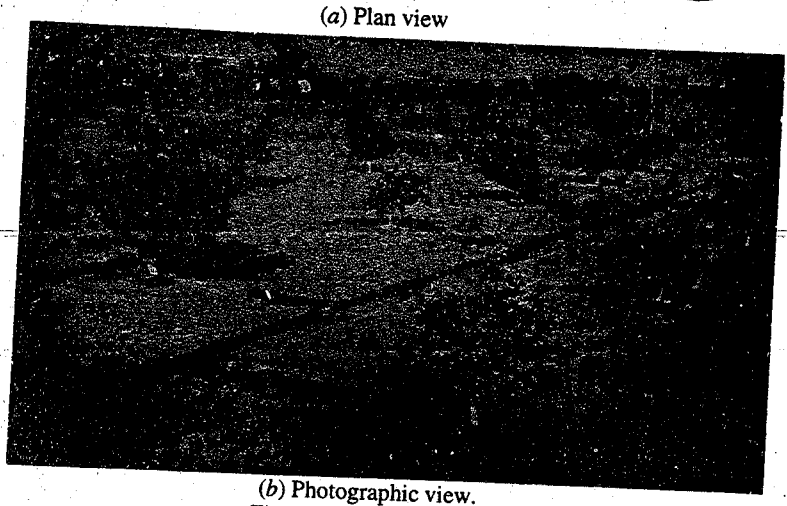
In check flooding, the *check* is filled with water at a fairly high rate and allowed to stand until the water infiltrates.

This method is suitable for more permeable soils as well as for less permeable soils. The water can be quickly spread in case of high permeable soils, thus reducing the percolation losses. The water can also be held on the surface for a longer time in case of less permeable soils, for assuring adequate penetration. These checks, are sometimes used to absorb water, where the stream-flow is diverted during periods of high run off.

(4) **Basin flooding.** This method is a special type of check flooding and is adopted specially for orchard trees. One or more trees are generally placed in the basin, and the surface is flooded as in check method, by ditch water, as shown in Fig. 1.5.



(*a*) Plan view

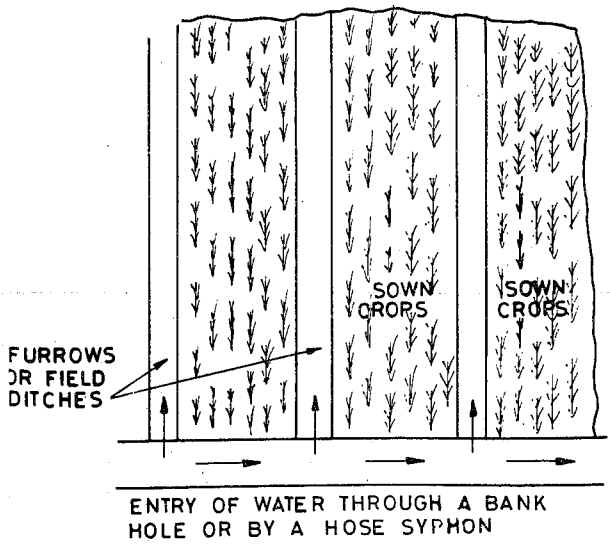


(*b*) Photographic view.

Fig. 1.5. Basin flooding.

(5) **Furrow irrigation method.** In flooding methods, described above, water covers the entire surface; while in furrow irrigation method (Fig. 1.6), only one-fifth to one-half of the land surface is wetted by water. It therefore, results in less evaporation, less pudding of soil, and permits cultivation sooner after irrigation.

Furrows are narrow field ditches, excavated between rows of plants and carry irrigation water through them. Spacing of furrows is determined by the proper spacing of the plants. Furrows vary from 8 to 30 cm deep, and may be as much as 400 metres long.



(*a*) Plan view.

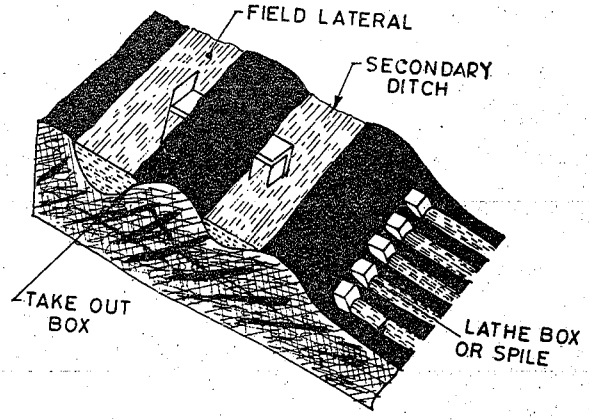


(*b*) Photographic view (using plastic syphons).
Fig. 1.6. Furrow Irrigation.

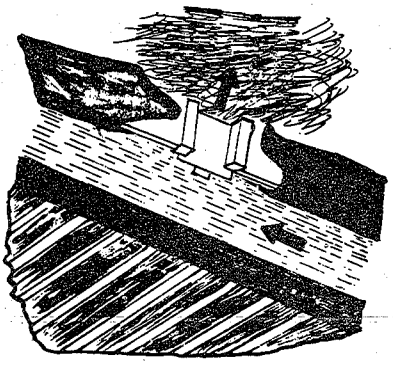
Excessive long furrows may result in too much percolation near the upper end, and too little water near the down-slope end. Deep furrows are widely used for row crops. Small shallow furrows called *corrugations*, are particularly suitable for relatively irregular topography and close growing crops, such as meadows and small grains.

Water may be diverted into the furrows by an opening in the bank of the supply ditch or preferably by using a rubber hose tubing, which can be primed by immersion in the ditch. The use of hose, prevents the necessity of breaking the ditch bank. and

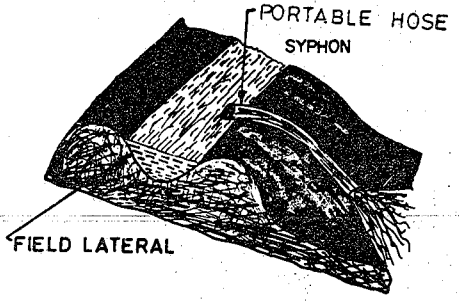
provides a uniform flow into the furrow. Fig. 1.7 shows such different devices which are used for distributing water into the fields from the supply ditch.



(*a*) Spiles or Lathe Box.



(*b*) Plastic Tube Syphons



(*c*) Border Takeout

Fig. 1.7. Devices for distributing water from irrigation ditches into fields.

(6) **Sprinkler irrigation method.** In this farm-water application method, water is applied to the soil in the form of a spray through a network of pipes and pumps. It is a kind of an artificial rain and, therefore, gives very good results. It is a costly process and widely used in U.S.A. It can be used for all types of soils and for widely different topographies and slopes. It can advantageously be used for many crops, because it fulfils the normal requirement of uniform distribution of water. This method possesses great potentialities for irrigating areas in Rajasthan in India, where other types of surface or sub-surface irrigation are very difficult.

Inspite of the numerous advantages which this method possesses over the other methods, it has not become popular in India for the simple reason that ours is a poor and a developing nation. This method is not only costly but requires a lot of technicalities. The correct design and efficient operation are very important for the success of this method. Special steps have to be taken for preventing entry of silt and debris, which are very harmful for the sprinkler equipment. Debris-choke nozzles, interfere with

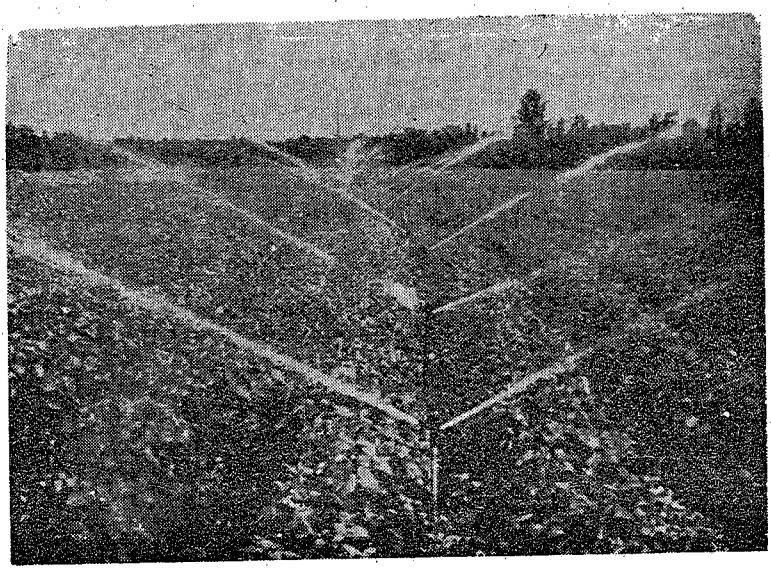


Fig. 1.8. Photoview of a sprinkler system in operation (using a rotary head).

the application of water on the land : while the abrasive action of silt causes excessive wear on pump impellers, sprinkler nozzles, and bearings. The system is to be designed in such a way that the entire sprayed water seeps into the soil, and there is no run off from the irrigated area.

The **conditions favouring the adoption of this method,** are :

(*i*) When the land *topography is irregular,* and hence unsuitable for surface irrigation.

(*ii*) When the land *gradient is steeper,* and soil is easily *erodible.*

(*iii*) When the land soil is *excessively permeable,* so as not to permit good water distribution by surface irrigation ; or when the soil is *highly impermeable.*

(*iv*) When the *watertable* is high.

(*v*) When the area is such that the seasonal water requirement is low, such as *near the coasts.*

(*vi*) When the *crops to be grown are such :*

(*a*) as to require humidity control, as in tobacco ;

(*b*) crops having shallow roots ; or

(*c*) crops requiring high and frequent irrigation.

(*vii*) When the water is available with difficulty and is scarce.

Types of sprinkler systems. A sprinkler system can be classified under three heads, as :

1. Permanent system ;
2. Semi-permanent system ; and
3. Portable system.

Earlier, the *fixed over-head perforated pipe installations* were being used for sprinkler network; but with the advent of light weight steel pipes and quick couplers, portable systems have been designed.

In *permanent system,* pipes are permanently buried in such a way that they do not interfere with the farming operations. In the *semipermanent system,* the main lines are buried in the ground, while the laterals are portable. In the *portable* system, the mains as well as laterals are portable. These portable networks can be moved from farm to farm.

In the sprinkler irrigation network, we have the mains and the sub-mains, through which water under pressure is made to flow. *Revolving sprinkler heads* are then usually mounted on *rising pipes* attached to the laterals. The water jet comes out through the revolving sprinkler heads, with force. When sprinkler heads are not provided, perforations are made in the pipes, and they are provided with nozzles, through which water jets out and falls on the ground. Generally, such a perforated pipe system operates at low heads ; whereas, the revolving head sprinklers operate on high as well as low heads, depending upon the type of rotary head used.

The **advantages** of sprinkler irrigation are enumerated below :

(*i*) Seepage losses, which occur in earthen channels of surface irrigation methods, are completely eliminated. Moreover, only *optimum quantity of water* is used in this method.

(*ii*) Land levelling is not required, and thus avoiding removal of top fertile soil, as happens in other surface irrigation methods.

(*iii*) No cultivation area is lost for making ditches, as happens in surface irrigation methods. It, thus, results in increasing about 16% of the cropped area.

(*iv*) In sprinkler system, the water is to be applied at a rate lesser than the infiltration capacity of the soil, and thus avoiding surface run off, and its bad effects, such as loss of water, washing of top soil, etc.

(*v*) Fertilisers can be uniformly applied, because they are mixed with irrigation water itself.

(*vi*) This method leaches down salts and prevents water-logging or salinity.

(*vii*) It is less labour oriented, and hence useful where labour is costly and scarce.

(*viii*) Upto 80% efficiency can be achieved, *i.e.* upto 80% of applied water can be stored in the root zone of plants.

The **limitations** of sprinkler irrigation are also enumerated below :

(*i*) High winds may distort sprinkler pattern, causing non-uniform spreading of water on the crops.

(*ii*) In areas of high temperature and high wind velocity, considerable evaporation losses of water may take place.

(*iii*) They are not suited to crops requiring frequent and larger depths of irrigation, such as paddy.

(*iv*) Initial cost of the system is very high, and the system requires a high technical skill.

(*v*) Only sand and silt free water can be used, as otherwise pump impellers lifting such waters will get damaged.

(*vi*) It requires larger electrical power.

(*vii*) Heavy soil with poor intake cannot be irrigated efficiently.

(*viii*) A constant water supply is needed for commercial use of equipment.

Note. The widely known Indian Commercial Company which specialises in installing sprinkler irrigation systems, is "Premier Irrigation Equipments (Pvt. Ltd.), Calcutta," with its Delhi office also. This firm and its publications can be referred to, for specialised knowledge in this branch of field irrigation.

(7) **Drip Irrigation Method.** Drip irrigation, also called **trickle irrigation,** is the latest field irrigation technique, and is meant for adoption at places where there exists acute scarcity of irrigation water and other salt problems. In this method, water is *slowly* and directly applied to the root zone of the plants, thereby minimising the losses by evaporation and percolation.

This system involves laying of a system of *head, mains, sub-mains, laterals,* and *drop nozzles.* Water oozes out of these small drip nozzzles uniformly and at a very small rate, directly into the plant roots area.

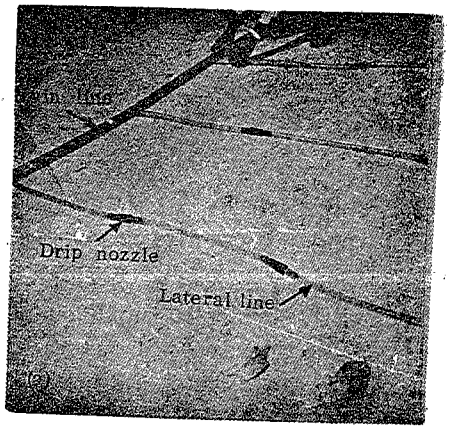
The head consists of a pump to lift water, so as to produce the desired pressure of about 2.5 atmosphere, for ensuring proper flow of water through the system. The lifted irrigation water is passed through a fertiliser tank, so as to mix the fertiliser directly in the irrigation water, and then through a filter, so as to remove the suspended particles from the water, to avoid clogging of drip nozzles.

The *mains and sub-mains* are the specially designed small sized pipes, made of flexible material like black PVC. These are generally buried or laid on the ground, as shown in Fig. 1.9. Their sizes should be sufficient to carry the design discharge of the system.

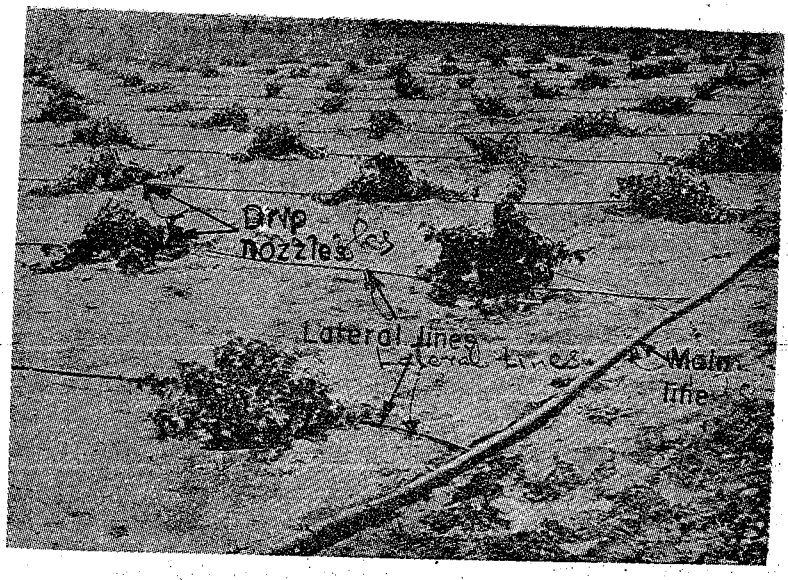
The *laterals* are very small sized (usually 1 to 1.25 cm dia.), specially designed, black PVC pipes, taking off from the mains or sub-mains, Laterals can usually be up to 50 m long, and one lateral line is laid for each row of crop. *Hardie Biwall* is a patented name of a special dual chambered micro tubing, manufactured from a linear low density polyethylene, and is being used these days for laterals.

The *drip nozzles,* also called *emitters,* or *valves,* are fixed on laterals, at regular intervals of about 0.5 to 1 m or so, discharging water at very small rates of the order of 2 to 10 litres per hour.

Like the sprinkler system, this method also involves specialised knowledge, and is not being adopted by our ordinary farmers. This method, is however, being used for small nurseries, orchards, or gardens. The widely known Commercial Indian Company, which specialises in this field irrigation method, is known as "Jain Irrigation Systems Ltd., Jalgaon" (Post Box No. 20), Pin Code : 425001. This firm can be contacted in special needs for layout of such an irrigation system.



(*a*) Layout of mains, laterals and drip nozzles.



(*b*) Field irrigated by drip system laid on the ground.

Fig. 1.9. Two photoviews of Drip irrigation system.

1.7. Quality of Irrigation Water

Just as every water is not suitable for human beings, in the same way, every water is not suitable for plant life. Water containing impurities, which are injurious to plant growth, is not satisfactory for irrigation, and is called the unsatisfactory water.

The quality of suitable irrigation water is very much influenced by the constituents of the soil which is to be irrigated. A particular water may be harmful for irrigation on a particular soil, but the same water may be tolerable or even useful for irrigation on some other soil. The various types of impurities, which make the water unfit for irrigation, are classified as :

(*i*) Sediment concentration in water

(*ii*) Total concentration of soluble salts in water

(*iii*) Proportion of sodium ions to other cations

(*iv*) Concentration of potentially toxic elements present in water

(*v*) Bicarbonate concentration as related to the concentration of calcium plus magnesium

(*vi*) Bacterial contamination.

The effects of these impurities are discussed below :

(1) **Sediment.** The effect of sediment present in the irrigation water depends upon the type of irrigated land. When fine sediment from water is deposited on sandy soils, the fertility is improved. On the other hand, if the sediment has been derived from the eroded areas, it may reduce the fertility or decrease the soil permeability. Sedimented water creates troubles in irrigation canals, as it increases their siltation and maintenance costs. In general, ground water or surface water from reservoirs, etc. does not have sufficient sediment to cause any serious problems in irrigation.

(2) **Total concentration of soluble salts.** Salts of calcium, magnesium sodium and potassium, present in the irrigation water may prove injurious to plants. When present in excessive quantities, they reduce the osmotic activities of the plants, and may prevent adequate aeration, causing injuries to plant growth. The injurious effects of salts on the plant growth depend upon the concentration of salts left in the soil.

The concentration of salts in water, may not appear to be harmful to the plants, but the concentration of salts which remain in the soil after the saline water is used up by the plants is much more than the first, and may prove to be harmful. In other words, at the beginning of irrigation with undesirable water, no harm may be evident, but with the passage of time, the salt concentration in the soil may increase to a harmful level, as the soil solution gets concentrated by evaporation.

Hence, the effects of salts on plant growth depend largely upon the total amount of salts present in the soil solution. The salinity concentration of the soil solution (C_s) after the consumptive water (C_u) has been extracted from the soil, is given by

$$C_s = \frac{C \cdot Q}{[Q - (C_u - P_{eff})]} \qquad ...(1.3)$$

where Q = The quantity of water applied

C_u = Consumptive use of water, *i.e.* the total amount of water used by the plant for its growth

P_{eff} = Useful rainfall

$C_u - P_{eff}$ = Used up irrigation water

C = Concentration of salt in irrigation water

CQ = Total salt applied to soil with Q amount of irrigation water.

The salt concentration is generally expressed by ppm (parts per million) or by mg/l (milligram per litre), both units being equal. The critical salt concentration in the irrigation water depends upon many factors, yet however, amounts in excess of 700 ppm are harmful to some plants, and more than 2000 ppm are injurious to all crops.

The salt concentration is generally measured by determining the electrical conductivity of water. They are directly proportional to each other. Electrical conductivity is expressed in micro mhos per centimetre. When its value is up to 250 micro mhos/cm at 25°C, it is called low conductivity water (Cl) ; when its value is between 250 to 750, it is called *Medium Conductivity water* (C2) ; when its value is between 750 to 2250, it is called *High Conductivity water* (C3) ; and the values above 2250, are classified as very *High Conductivity water* (C4). The suitabilities of these four types of waters for irrigation supplies are discussed in Table 1.1. This classification which is shown in Table 1.1 was given by U.S.D.A. Handbook No. 60 (1954).

Table 1.1

S. No.	*Type of water*	*Use in irrigation*
1.	Low salinity water (C1). Conductivity between 100 to 250 micro mhos/cm at 25°.	Can be used for irrigation for almost all crops and for almost all kinds of soils. Very little salinity may develop, which may require slight leaching; but it is permissible under normal irrigation practices except in soils of extremely low permeabilities.
2.	Medium salinity water (C2). Conduc- tivity between 250 to 750 micro mhos/cm at 25°C.	Can be used, if a moderate amount of leaching occurs. Normal salt-tolerant plants can be grown without much salinity control.
3.	High salinity water (C3). Conductivity between 750 to 2250 micro mhos/cm at 25°C.	Cannot be used on soils with restricted drainage. Special precautions and measures are undertaken for salinity control and only high-salt tolerant plants can be grown.
4.	Very high salinity water (C4). Conductivity more than 2250 micro mhos/cm at 25°C.	Generally not suitable for irrigation.

(3) **Proportion of sodium ions to other cations.** Most of the soils contain calcium and magnesium ions and small quantities of sodium ions. *The percentage of the sodium ions is generally less than 5% of the total exchangeable cations.* If this percentage increases to about 10% or more, the aggregation of soil grains breaks down. The soil becomes less permeable and of poorer tilth. It starts crusting when dry and its pH increases towards that of an alkaline soil. High sodium soils are, therefore, plastic, sticky when wet, and are prone to form clods, and they crust on drying.

The proportion of sodium ions present in the soils, is generally measured by a factor called *Sodium-Absorption Ratio* (SAR) and represents the sodium hazards of water. SAR is defined as :

$$SAR = \frac{Na^{+}}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} \qquad ...(1.4)$$

where, the concentration of the ions is expressed in equivalent per million (epm) ; epm is obtained by dividing the concentration of salt in mg/l or ppm by its combining weight (*i.e.* Atomic wt. ÷ Valence.)

When the value of SAR lies between 0 to 10, it is called *Low Sodium Water* (S1) ; when between 10 to 18, it is called *Medium Sodium Water* (S2) ; between 18 to 26 ; it is called *High Sodium Water* (S3) ; and when values of SAR are more than 26,it is called

Very High Sodium Water (S4). The suitabilities of these four kinds of water for irrigation are discussed in Table 1.2. The SAR value can be reduced by adding gypsum ($CaSO_4$) to the water or to the soil.

Table 1.2

S. No.	*Type of water*	*Use in irrigation*
1.	Low sodium water (S1). SAR value lying between 0 to 10.	Can be used for irrigation on almost all soils and for almost all crops except those which are highly sensitive to sodium, such as stone-fruit trees and adocados, etc.
2.	Medium sodium water (S2). SAR value lying between 10 to 18.	Appreciably hazardous in fine textured soils, which may require gypsum, etc. ; but may be used on course-textured or organic soils with good permeability.
3.	High sodium water (S3). SAR value lying between 18 to 26.	May prove harmful on almost all the soils, and do require good drainage, high leaching, gypsum addition etc. for proper irrigation.
4.	Very high sodium water (S4). SAR value above 26.	Generally, not suitable for irrigation.

Depending upon the *Electrical conductivity EC* (representing salt content) of water, the *exchangeable sodium percentage ESP* (representing percentage of sodium w.r.to total exchangeable cations), and the pH value of the soil, the soils are classified as *saline, alkaline*, or *saline- alkali*, as shown in table 1.3 below :

Table 1.3. The Classification of Saline and Alkaline Soils

Sl. No.	*Classification*	*Electrical Conductivity (EC) in micro-mho/cm*	*Exchangeable Sodium Percentage (ESP)*	*pH value*
1.	**Saline soil** or *white alkali*	> 4000	< 15	≤ 8.5
2.	**Alkaline soil** or *Non-saline alkali* or *Sodic soil* or *Black alkali*	< 4000	> 15	8.5 to 10.0
3.	*Saline-alkali soil*	> 4000	> 15	< 8.5

(4) **Concentration of potentially toxic elements.** A large number of elements such as boron, selenium, etc. may be toxic to plants. Traces of *Boron* are essential to plant growth, but its concentrations above 0.3 ppm may prove toxic to certain plants. The concentration above 0.5 ppm is dangerous to nuts, citrus fruits and *deciduous* fruits. Cotton, Cereals and certain truck crops are moderately tolerant to boron, while Dates, Beets, Asparagus etc. are quite tolerant. Even for the most tolerant crops, the boron concentration should not exceed 4 ppm. Boron is generally present in various soaps. The waste water containing soap, etc. should, therefore, be used with great care in irrigation.

Selenium, even in low concentration, is toxic, and must be avoided.

(5) **Bicarbonate concentration as related to concentration of calcium plus magneisum.** High concentration of bi-carbonate ions may result in precipitation of calcium and magnesium bicarbonates from the soil-solution, increasing the relative proportion of sodium ions and causing sodium hazards.

(6) **Bacterial contamination.** Bacterial contamination of irrigation water is not a serious problem, unless the crops irrigated with highly contaminated water are directly

eaten, without being cooked. Cash crops like cotton, nursery stock, etc. which are processed after harvesting, can, therefore, use contaminated waste waters, without any trouble.

Example 1.1. *(a) What is the classification of irrigation water having the following characteristics : Concentration of Na, Ca and Mg are 22, 3 and 1.5 milli-equiavalents per litre respectively, and the electrical conductivity is 200 micro mhos per cm at 25°C ? (b) What problems might arise in using this water on fine textured soils ? (c) What remedies do you suggest to overcome this trouble ?*

Solution.

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{++} + Mg^{++}}{2}}} = \frac{22}{\sqrt{\frac{3+1.5}{2}}} = \frac{22}{\sqrt{2.25}} = \frac{22}{1.5} = 14.67.$$

If SAR is between 10 to 18, then it is classified as medium Sodium water and is represented by S2 (See Table 1.2).

Electrical conductivity is 200 micro-mhos per cm at 25°C.

According to Table 1.1, the water is called of Low conductivity (C1) if the value of electrical conductivity is between 100 to 250 micro mhos per cm at 25°C. It is, therefore, C1 water.

Hence, the given water is classified as C1-S2 *water.* **Ans.**

(*b*) In fine-textured soils, the medium sodium (S2) water may create the following problems :

(*i*) Soil becomes less permeable.

(*ii*) It starts crusting when dry.

(*iii*) It becomes plastic and sticky when wet.

(*iv*) Its pH increases towards that of alkaline soil.

(*c*) Gypsum ($CaSO_4$) addition, either to soil or to water is suggested to overcome sodium hazards posed by the given water.

PROBLEMS

1. Define irrigation and explain its necessity in a tropical country like India. What are the advantages and ill-effects of assured irrigation ?

2. What is meant by surface and sub-surface irrigation ; and what are their types ? Discuss briefly the various techniques used for distributing water in the farms.

3. What is meant by 'Furrow Irrigation" and 'Sprinkler Irrigation' ? Which one is preferred in India and why ?

4. 'The sprinkler system of irrigation is an excellent method but not used in India'. Discuss critically and briefly.

5. What is meant by 'Border flooding', and how does it differ from 'Check flooding' and 'Free flooding'?

6. Describe briefly the necessity and importance of irrigation works in our country? What are different types of irrigation ? Write brief notes on each of them.

7. (*a*) What are the benefits that can be accrued from Irrigation projects ?

(*b*) What is 'flood irrigation' ? Where is it practised ?

(*c*) How is the Flow irrigation different from the Lift irrigation ? Give the names of districts in Tamil Nadu where they are practised mostly ?

8. Discuss critically the quality standards required for irrigation water.

9. 'All the waters are not fit for irrigating crops'. Discuss briefly and critically the above statement.

10. What is meant by C2—S2 water ? Discuss its usefulness for irrigating fine textured soils.

11. Write short notes on :

(*i*) Lift irrigation.

(*ii*) Mixed cropping.

(*iii*) Ill-effects of irrigation.

(*iv*) Border strip and Sprinkler methods of irrigating fields.

(*v*) Sodium-Absorption-Ratio (SAR).

(*vi*) Salt concentration of irrigation waters and their utility in irrigation.

(*vii*) Sodium hazards of irrigation waters.

(*viii*) Boron concentration in irrigation waters.

(*ix*) Drip irrigation method.

2

Water Requirements of Crops

2.1. General

Every crop requires a certain quantity of water after a certain fixed interval, throughout its period of growth. If the natural rain is sufficient and timely so as to satisfy both these requirements, no irrigation water is required for raising that crop. In England, for example, the natural rain falling regularly throughout the year, satisfies both these requirements for practically all the crops, and, therefore, irrigation is not significantly needed in England. But in a tropical country like India, the natural rainfall is either insufficient, or the water does not fall regularly, as required by the crops. Since the magnitude as well as the frequency of the rainfall varies throughout a tropical country, certain crop may require irrigation in certain part of the country, and the same crop may not require any irrigation in some other part of the country. The area where irrigation is a must for agriculture is called the **arid region,** while the area in which inferior crops can be grown without irrigation is called a **semi-arid region.**

The term 'Water requirements of a crop' means *the total quantity and the way in which a crop requires water, from the time it is sown to the time it is harvested.* It is very clear from the above discussion, that the water requirement, will vary with the crop as well as with the place. In other words, *different crops will have different water requirements, and the same crop may have different water requirements at different places of the same country ;* depending upon the variations in climates, type of soils, methods of cultivation, and useful rainfalls, etc.

2.2. Crop Period or Base Period

The time period that elapses from the instant of its sowing to the instant of its harvesting is called the *crop-period.* The time between the first watering of a crop at the time of its sowing to its last watering before harvesting is called the *Base period* or the *Base of the crop.* Crop period is slightly more than the base period, but for all practical purposes, they are taken as one and the same thing, and generally expressed in days. Hence, in future, the terms like *growth period, crop period, base period,* etc., will be used as synonyms, each representing crop period, and will be represented by B (in days).

2.3. Duty and Delta of a Crop

2.3.1. Delta. Each crop requires a certain amount of water after a certain fixed interval of time, throughout its period of growth. The depth of water required every time, generally varies from 5 to 10 cm depending upon the type of the crop, climate and soil. The time interval between two such consecutive waterings is called the **frequency of irrigation,** or **rotation period.** The rotation period may vary between 6-15 days for

different crops. The summation of the total water depth supplied during the base period of a crop, for its full growth, will evidently represent the total quantity of water required by the crop for its full-fledged nourishment. *This total quantity of water required by the crop for its full growth* (maturity) may be expressed in hectare-metre (Acre-ft) or in million cubic metres (million cubic-ft) or simply as depth to which water would stand on the irrigated area, if the total quantity supplied were to stand above the surface without percolation or evaporation. *This total depth of water (in cm) required by a crop to come to maturity is called its delta* (Δ).

Example 2.1. *If rice requires about 10 cm depth of water at an average interval of about 10 days, and the crop period for rice is 120 days, find out the delta for rice.*

Solution. Water is required at an interval of 10 days for a period of 120 days. It evidently means that 12 no. of waterings are required, and each time, 10 cm depth of water is required. Therefore, total depth of water required

$$\Delta = 12 \times 10 \text{ cm} = 120 \text{ cm}.$$

Hence Δ for rice = 120 cm. **Ans.**

Example 2.2. *If wheat requires about 7.5 cm of water after every 28 days, and the base period for wheat is 140 days, find out the value of delta for wheat.*

Solution. Assuming the base period to be representing the crop period, as per usual practice, we can easily infer that the water is required at an average interval of 28 days up to a total period of 140 days. This means that $\frac{140}{28} = 5$ no. of waterings are required.

The depth of water required each time = 7.5 cm.

$\therefore$ Total depth of water reqd. in 140 days = 5×7.5 cm = 37.5 cm

Hence, Δ for wheat = 37.5 cm. **Ans.**

2.3.2. Delta for certain crops. The average values of deltas for certain crops are shown in Table 2.1. These values represent the total water requirement of the crops. The actual requirement of irrigation water may be less, depending upon the useful rainfall. Moreover, these values represent the values on field, *i.e.* 'delta on field' which includes the evaporation and percolation losses.

Table 2.1. Average Approximate Values of Δ for Certain Important Crops in India

S. No. (1)	*Crop* (2)	*Delta on field* (3)
1.	Sugarcane	120 cm (48″)
2.	Rice	120 cm (48″)
3.	Tobacco	75 cm (30″)
4.	Garden fruits	60 cm (24″)
5.	Cotton	50 cm (22″)
6.	Vegetables	45 cm (18″)
7.	Wheat	40 cm (16″)
8.	Barley	30 cm (12″)
9.	Maize	25 cm (10″)
10.	Fodder	22.5 cm (9″)
11.	Peas	15 cm (6″)

2.3.3. Duty of Water. The 'duty' of water is the relationship between the volume of water and the area of the crop it matures. It may be defined as the number of hectares of land irrigated for full growth of a given crop by supply of 1 m^3/sec of water continuously during the entire base period (B) of that crop. Thus, if water flowing at a rate of one cubic metre per second, runs continuously for B days, and matures 200 hectares, then the duty of water for that particular crop will be defined as 200 hectares per cumec to the base of B days. The duty is generally represented by the letter D.

2.3.4. Relation between duty and delta. Let there be a crop of base period B days. Let one cumec of water be applied to this crop on the field for B days.

Now, the volume of water applied to this crop during B days

$$= V = (1 \times 60 \times 60 \times 24 \times B)\ m^3$$
$$= \mathbf{86\,400\,B}\ \text{(cubic metre)}$$

By definition of duty (D), one cubic metre supplied for B days matures D hectares of land.

$\therefore$ This quantity of water (V) matures D hectares of land or $10^4 D$ sq. m of area.

Total depth of water applied on this land

$$= \frac{\text{Volume}}{\text{Area}} = \frac{86{,}400\,B}{10^4\,D} = \frac{8.64\,B}{D}\ \text{metres}$$

By definition, this total depth of water is called delta (Δ).

$$\therefore \quad \Delta = \frac{8.64\,B}{D}\ \text{metres} \qquad \text{...(2.1)}$$

or
$$\Delta = \frac{864\,B}{D}\ \text{cm.} \qquad \text{...(2.2)}$$

where, Δ is in cm, B is in days ; and
D is duty in hectares/cumec.

Example 2.3. *Find the delta for a crop when its duty is 864 hectares/cumec on the field, the base period of this crop is 120 days.*

Solution.

$$\Delta\ (\text{cm}) = \frac{864\,B}{D}$$ where B is in days and D is in hectares/cumec

In this question, $B = 120$ days and $D = 864$ hectares/cumec

$$\therefore \quad \Delta = \frac{864 \times 120}{864} = \mathbf{120\ cm.} \quad \textbf{Ans.}$$

2.3.5. Duty at various places. In a large canal irrigation system, the water from its source, first of all, flows into the main canal ; from the main canal, it flows into the branch canal ; from the branch canal, it flows into the distributary ; from the distributary, it flows into the minor ; and then into the field channels (water-courses) ; and finally into the fields. A systematic layout of a canal system is shown in Fig. 2.1.

During the passage of water from these irrigation channels, water is lost due to evaporation and percolation. These losses are called Transit losses or Transmission or Conveyance losses in channels.

Duty of water for a crop, is the number of hectares of land, which the unit flow of water for B days can irrigate. Therefore, if the water requirement of a particular crop at a particular location is more, then lesser number of hectares of land it will irrigate. Hence, *if water consumed by a crop of a given base period is more, its duty will be less. It, therefore, becomes clear that the duty of water at the head of the water-course will be less than the duty of water 'on the field' ; because when water flows from the head of the water-course and reaches the field, some water is lost en-route as transit losses.* Applying the same reasoning, it can be established *that duty of water at the head of a minor will be less than that at the head of the water-course ; duty at the head of a distributary will be less than that at the head of a minor, duty at the head of a branch canal will be less than that at the head of a distributary, and duty at the head of a main canal will be less than the duty at the head of a branch canal.* Duty of water, therefore, varies from one place to another, and *increases* as one moves *downstream* from the head of the main canal towards the head of the branches or water-courses. *The duty at the head of water-course (i.e.* at the outlet point of the minor), is quite important, and is called the **outlet discharge factor.** This outlet point is generally the end point of Irrigation Department. The control of Irrigation Department finishes at the outlet point, and the water is carried into the fields through water-courses by the beneficiary cultivators themselves.

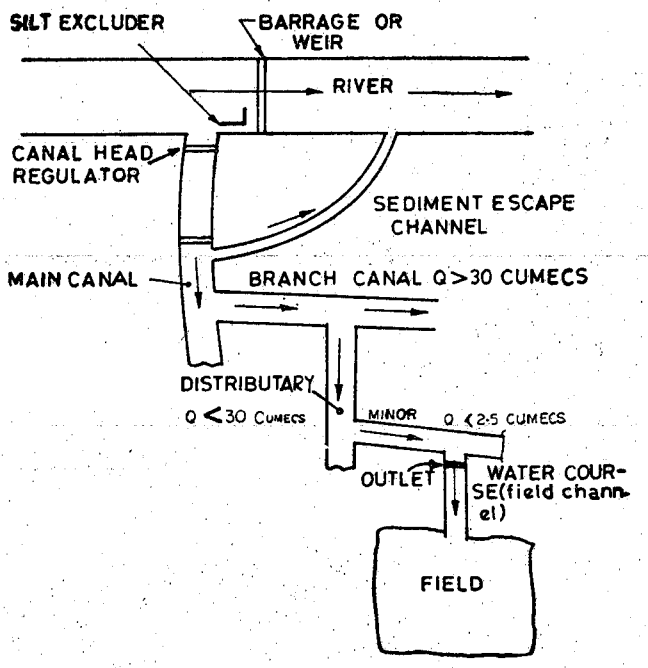


Fig. 2.1. Layout of a canal system.

2.3.6. Flow duty and Quantity duty. In direct irrigation, duty is always expressed in hectares/cumec. It is then called as *flow-duty* or duty.

In storage irrigation, duty may, sometimes be expressed in hectares/million cubic metre of water available in the reservoir. It eventually means that every million cubic metre of water available in the reservoir will mature so many hectares of a particular crop. Hence, the irrigation capacity of the reservoir is directly known. When duty is expressed in this manner, it is called *Quantity duty* or *Storage duty.*

2.3.7. Factors on which duty depends. Duty of irrigation water depends upon the following factors :

(*i*) *Type of crop.* Different crops require different amount of water, and hence, the duties for them are different. A crop requiring more water will have less flourishing acreage for the same supply of water as compared to that requiring less water. Hence, *duty will be less for a crop* requiring *more* water and *vice versa.*

(*ii*) *Climate and season.* As stated earlier, duty includes the water lost in evaporation and percolation. These losses will vary with the season. Hence, duty varies from season to season, and also from time to time in the same season. The figures for duties which we generally express are their average values considered over the entire crop period.

(*iii*) *Useful rainfall.* If some of the rain, falling directly over the irrigated land, is useful for the growth of the crop, then so much less irrigation water will be required to mature the crop. More the useful rainfall, less will be the requirement of irrigation water, and hence, more will be the duty of irrigation water.

(*iv*) *Type of soil.* If the permeability of the soil under the irrigated crop is high, the water lost due to percolation will be more and hence, the duty will be less. Therefore, for sandy soils, where the permeability is more, the duty of water is less.

(*v*) *Efficiency of cultivation method.* If the cultivation method (including tillage and irrigation) is faulty and less efficient, resulting in the wastage of water, the duty of water will naturally be less. If the irrigation water is used economically, then the duty of water will improve, as the same quantity of water would be able to irrigate more area. Cultivators should, therefore, be trained and educated properly to use irrigation water economically.

2.3.8. Importance of duty. It helps us in designing an efficient canal irrigation system. Knowing the total available water at the head of a main canal, and the overall duty for all the crops required to be irrigated in different seasons of the year, the area which can be irrigated can be worked out. Inversely, if we know the crops area required to be irrigated and their duties, we can work out the discharge required for designing the channel.

2.3.9. Duty for certain crops. The average values of duties for certain important Indian crops are tabulated in Table 2.2.

Table 2.2. Average Approximate Values of Duty for Certain Important Crops in India

Crop	*Duty in hectares/cumec*
Sugarcane	730
Rice	775
Other Kharif	1500
Rabi	1800
Perennials	1100
Hot fodder	2000

2.3.10. Measures for Improving duty of water. The duty of canal water can certainly be improved by effecting economy in the use of water by resorting to the following precautions and practices.

(1) *Precautions in field preparation and sowing :*

(*i*) Land to be used for cultivation should, as far as possible, be levelled.

(*ii*) The fields should be properly ploughed to the required depth.

(*iii*) Improved modern cultivation methods may preferably be adopted.

(*iv*) Porous soils should be treated before sowing crops to reduce seepage of water.

(*v*) Alkaline soils should be properly leached before sowing.

(*vi*) Manure fertilisers should be added to increase water holding capacity of the soil.

(*vii*) *Rotation of crops** should be preferred, as this will ensure increased crop yields with minimum use of water.

(2) *Precautions in handling irrigation supplies :*

(*i*) The source of irrigation water should be situated within the prescribed limits, and should be capable of delivering sufficient quantity of satisfactary quality of irrigation water.

(*ii*) Canals carrying irrigation supplies should be lined to reduce seepage and evaporation**, thereby reducing on field requirement of water and consequently improving the duty of water.

(*iii*) Water courses may preferably be lined ; or R.C.C. pipes may be used for the same to reduce on field requirement of water, thereby improving duty.

(*iv*) Free flooding of fields should be avoided and furrow irrigation method may preferably be adopted, if surface irrigation is resorted to.

(*v*) Sub surface irrigation and Drip irrigation may be preferred to ordinary surface irrigation.

(*vi*) If canals are not lined, then two canals running side by side may be preferred to a single canal, as this will reduce the FSL, thereby reducing percolation losses.

(*vii*) Irrigation supplies should be economically used by proper control on its distribution, volumetric assessment, and by imparting proper education to the farmers.

2.4. Crop Seasons and Indian Agriculture

More than 70% of the Indian population is directly or indirectly connected with agriculture. The chief crops of India are rice, wheat, sugarcane, tea, cotton, groundnut, jute, coffee, rubber, garden crops (like coconuts, orange, etc), etc. Different types of soils are needed for raising different types of crops. For example, heavy retentive soil (40% clay) is favourable for raising crops like sugarcane, rice, etc., requiring more water. Light sandy soil (2 to 8% clay) is suitable for crops like gram, fodder, etc. requiring less water. Medium or normal soil (having about 10—20% of clay) is suitable for crops like wheat, cotton, maize, vegetables, oil seeds, etc. requiring normal amount of water.

From the agricultural point of view, the year can be divided into two principal cropping seasons, *i.e. Rabi* and *Kharif.* Normally, **Rabi** *starts from 1st October and ends on 31st March ; while* **Kharif** *starts from 1st April and ends on 30th September.* These dates are not rigid dead lines. The time may vary up to 1—3 months on either side. Sugarcane, which is an important cash crop, extends over both seasons.

The **Kharif crops** *are rice, bajra, jowar, maize, cotton, tobacco, groundnut, etc. The* **Rabi crops** *are wheat, barley, gram, linseed, mustard, potatoes, etc.* Kharif crops are also called 'Summer crops' and Rabi crops as 'Winter crops'. Kharif crops require about two to three times the quantity of water required by the Rabi crops.

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* explained in article 2.5.5.

** Water moving with higher velocity, will ensure reduced evaporation.

The above distinction of seasons is well applicable to North India, but in South India, there is no such marked distinction between the different seasons. In fact, in South India, there is no clear cut winter, spring, summer and autumn seasons, as they are in North India. Except Bombay-Deccan, where there are five crop seasons, there are only three crop seasons in the remaining parts of the country. These three classifications of seasons are :

(*i*) Hot weather or Kharif season. (*ii*) Monsoon season. (*iii*) Winter or Rabi season.

When a crop requires water for its crop season and also for some time in the beginning of the next crop season, allowance has to be made for this overlap. This allowance is known as **overlap allowance,** Sugarcane is an example of this kind of crop. Some important Indian crops, their periods of growth, water requirements, seed requirements, yields, etc. are shown in Table 2.3.

Table 2.3. Irrigation Requirements of Certain Important Indian Crops

S. No.	*Crop*	*Period of growth*	*Av. water depth reqd. (in cm)*	*Irrigation requirements and remarks*	*Average quantity of seed required in kg/hectare*	*Average quantity of yield obtained in kg/hectare*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
	Kharif Crops					
(*i*)	Maize, high yielding	June to Sept.-Oct.	45	Four or five waterings. Sensitive to drought and floods. Responsible to fertilizers.	15	3,000
(*ii*)	Bajra (spiked millets, or Pearl millets), high yielding	July to Nov.	30	Water should not stand. Irrigation as required. Resistant to drought and flooding.	3.75	2,000
(*iii*)	Juar (Great millets), high yielding	Sown in July as fodder and cut green more than once.	30	Same as above.	12.5	3,000
(*iv*)	Ground-nut	May to Nov-Dec.	45	'Paleo' reqd. before sowing.	—	1,600
(*v*)	Cotton	May–June to Nov.–Jan.	25—40	Three or four irrigations are required. Damage up to the extent of 50% may be caused by flooding, rains, etc.	—	500
(*vi*)	Pulses like Arhar, etc.	July-Aug. to Nov.-Dec.	30	Irrigated when leaves get dried.	12.5	700
(*vii*)	Transplanted Rice (Paddy), high yielding	July to Nov.	125—150	Standing water of 5 to 8 cm gives best results.	30 to 35 kg of seed is sufficient to raise nursery to transplant one hectare	4500
(*viii*)	Til	July-Aug. to Oct.-Nov.	—	Generally not irrigated but better to irrigate once.	1.25	350

(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Rabi Crops					
(i)	Wheat (ordinary)	Oct. to March–April	37.5	Three-four waterings of 7–10 cm depth.	80—100	1500
(ii)	High yielding Wheat	Oct. to March–April	45	Five-six waterings of 7—10 cm depth.	100—125	4000
(iii)	Gram (high yielding)	Sept.-Oct. to March	30	Irrigated when leaves get dry.	12.5	3500
(iv)	Barley	Oct. to Mar.-April	30	Two waterings ; one at jointing and another at booting stage.	120	1300
(v)	Potatoes	Sept.-Oct. to Feb.	60—90	Usually irrigated ; sown in high hills up to early April. Second crop in plains is sometimes, taken in Feb.-april.	15,000	35,000
(vi)	Tobacco	Oct.-Feb. to Feb.-May	60	Four to five waterings.		4,500
(vii)	Linseed *i.e.* Alsi	Oct.-Nov. to March	45—50	Irrigated at intervals of 15 days. Resistant to drought but damaged by frost and flooding.		700
(viii)	Mustard	Oct. to Feb.-Mar.	45	Watered at intervals of 7—10 days.	33	1000 to 1600
	Overlapping crop but generally classified under Rabi crop					
(i)	Sugarcane	Feb.-March to Dec.-March	90	5 or 6 waterings of 10 cm or more.	500	25,000—30,000

2.5. Certain Important Definitions

2.5.1. Kharif-Rabi ratio or Crop ratio. The area to be irrigated for Rabi crop is generally more than that for the Kharif crop. This ratio of proposed areas, to be irrigated in Kharif season to that in the Rabi season is called, Kharif-Rabi ratio. This ratio is generally 1 : 2, *i.e.* Kharif area is one-half of the Rabi area.

2.5.2. Paleo irrigation. Sometimes, in the initial stages before the crop is sown, the land is very dry. This particularly happens at the time of sowing of Rabi crops because of hot September, when the soil may be too dry to be sown easily. In such a case, the soil is moistened with water, so as to help in sowing of the crops. This is known as Paleo irrigation.

2.5.3. Kor-watering. The first watering which is given to a crop, when the crop is a few centimetres high, is called kor-watering. It is usually the maximum single watering followed by other waterings at usual intervals, as required by drying of leaves.

The optimum depth of kor-watering for different crops are different. *For example, the optimum depth of kor-watering for Rice is 19 cm., for Wheat (in U.P.) is about 13.5 cm, and for Sugarcane is 16.5 cm.*

The kor-watering must be applied within a fixed limited period, called *Kor-period.* If the plants fail to receive this water in time or in sufficient quantity, then they do suffer a significant loss. The kor-period depends upon the climate. It is less for humid climates

and more for dry climates. The *kor-period for rice varies from 2 to 4 weeks, and that for wheat varies from 3 to 8 weeks.*

2.5.4. Cash crops. A cash crop may be defined as a crop which has to be encashed in the market for processing, etc. as it cannot be consumed directly by the cultivators. All non-food crops, are thus, included in cash crops. Crops like jute, tea, cotton, tobacco, sugarcane, etc. are, therefore, called cash crops. The food crops like wheat, rice, barley, maize, etc. are excluded from the list of cash crops.

2.5.5. Crops rotation. When the same crop is grown again and again in the same field, the fertility of land gets reduced as the soil becomes deficient in plant foods favourable to that particular crop. In order to enhance the fertility of the land and to make the soil regain its original structure, it is often found necessary and helpful to give some rest to the land. This can be achieved either by allowing the land to lie fallow without any cultivation for some time, or to grow crops which do not mainly require those salts or foods which were mainly required by the earlier grown crop. This method of growing different crops in rotation, one after the other, in the same field, is called *Rotation of Crops.* A cash crop may be followed by a fodder crop, which, in turn, may be followed by a *soil-renovating crop* like gram, which being a **liguminous crop**, helps in giving nitrogen to the fields, thereby renovating the soil. The cultivators who are fond of sowing cash crops always, should be educated and made to understand the advantages of sowing crops in rotation.

The rotation of crops will help in extracting different foods from the soil, and thus avoiding the general deficiency of any particular type(s) of element(s). Moreover, if only one type of crop is grown in the same field, numerous insects and pests (of similar nature) will get developed. The crop rotation will also help in checking such growths. *Crop rotation will thus help in increasing the fertility of soil, and reducing the diseases and wastage due to insects, and hence increasing the overall crop yield.*

In general, the following rotations of crops may be adopted depending upon the soil conditions :

(*i*) Wheat—Juar—Gram

(*ii*) Rice—Gram*

(*iii*) Cotton—Wheat—Gram*— Fallow (up to July)

(*iv*) Cotton—Juar—Gram.

(*v*) Sugarcane (18 months) — Thadwa — Wheat or gram — Fallow (upto July).

2.6. Optimum Utilisation of Irrigation Water

If a crop is sown and produced under absolutely identical conditions, using different amounts of water depths, the yield is found to vary. The yield increases with water, reaches a certain maximum value and then falls down, as shown in Fig. 2.2. *The quantity of water at which the yield is maximum, is called the optimum water depth.*

Therefore, optimum utilisation of irrigation, generally means, getting maximum yield with any amount of water. The supplies of water to the various crops should be adjusted in such a fashion, as to get optimum benefit ratio, not only for the efficient use of available water and maximum yield, but also to prevent water-logging of the land

* After harvesting a heavy water consuming crop like Rice, a less water consuming short term crop like gram may be taken, which may come up on the remnant moisture and manure.

in question. To achieve economy in the use of water, it is necessary that the farmers be made acquainted with the fact that only a certain fixed amount of water gives best results. More than that quantity, as well as, less than that quantity, reduces the yield. Many cultivators, till today feel, that they can increase the crop yield by using more and more water. Hence, they try to supply more water to their fields by undue tapping at the outlets. This must be checked. Moreover, farmers should be encouraged to line their water courses, thereby saving at least 20% of the costly irrigation water, which can be used to irrigate extra additional fields.

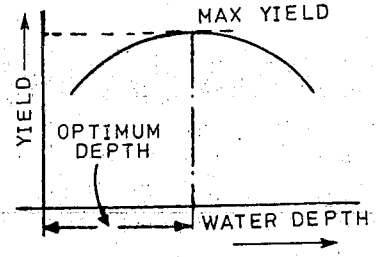


Fig. 2.2.

2.7. Irrigation Efficiencies

Efficiency is the ratio of the water output to the water input, and is usually expressed as percentage. Input minus output is nothing but losses, and hence, if losses are more, output is less and, therefore, efficiency is less. Hence, efficiency is inversely proportional to the losses. Water is lost in irrigation during various processes and, therefore, there are different kinds of irrigation efficiencies, as given below :

(*i*) **Efficiency of water-conveyance.** It is the ratio of the water delivered into the fields from the outlet point of the channel, to the water entering into the channel at its starting point. It may be represented by η_c. It takes the conveyance or transit losses into consideration.

(*ii*) **Efficiency of water-application.** It is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water actually delivered into the field. It may be represented by η_a. It may also be called **on farm efficiency,** as it takes into consideration the water lost in the farm.

(*iii*) **Efficiency of water-storage.** It is the ratio of the water stored in the root zone during irrigation to the water needed in the root zone prior to irrigation (*i.e.* field capacity – existing moisture content). It may be represented by η_s.

(*iv*) **Efficiency of water use.** It is the ratio of the water beneficially used, including leaching water, to the quantity of water delivered. It may be represented by η_u.

Example 2.4. *One cumec of water is pumped into a farm distribution system. 0.8 cumec is delivered to a turn-out, 0.9 kilometre from the well. Compute the conveyance efficiency.*

Solution. By definition,

$$\eta_c = \frac{\text{Output}}{\text{Input}} \times 100 = \frac{0.8}{1.0} \times 100 = 80\% \quad \textbf{Ans.}$$

Example 2.5. *10 cumecs of water is delivered to a 32 hectare field, for 4 hours. Soil probing after the irrigation indicates that 0.3 metre of water has been stored in the root zone. Compute the water application efficiency.*

Solution. Volume of water supplied by 10 cumecs of water applied for 4 hours

$$= (10 \times 4 \times 60 \times 60)\ m^3 = 1,44,000\ m^3$$
$$= 14.4 \times 10^4\ m^3 = 14.4\ m \times 10^4\ m^2 = 14.4\ ha.m.$$

($\because$ $10^4\ m^2 = 1$ hectare)

$\therefore$ Input = *14.4 ha.m* ...(*i*)

Output = 32 hectares land is storing water upto 0.3 m depth.

$\therefore$ Output = 32×0.3 ha.m = *9.6 ha.m* ...(*ii*)

$$\text{Water application efficiency } (\eta_a) = \frac{\text{Output}}{\text{Input}} \times 100 = \frac{9.6}{14.4} \times 100 = \mathbf{66.67\%}$$ **Ans.**

(*v*) **Uniformity coefficient or Water distribution efficiency**. The effectiveness of irrigation may also be measured by its water distribution efficiency (η_d), which is defined below :

$$\eta_d = \left(1 - \frac{d}{D}\right) \quad \text{...(2.3)}$$

where η_d = Water distribution efficiency.

D = Mean depth of water stored during irrigation.

d = Average of the absolute values of deviations from the mean.

The water distribution efficiency represents the extent to which the water has penetrated to a uniform depth, throughout the field. When the water has penetrated uniformly throughout the field, the deviation from the mean depth is zero and water distribution efficiency is 1.0.

Example 2.6. *The depths of penetrations along the length of a boarder strip at points 30 metres apart were probed. Their observed values are 2.0, 1.9, 1.8, 1.6 and 1.5 metres. Compute the water distribution efficiency.*

Solution. The observed depths at five stations are 2.0, 1.9, 1.8, 1.6 and 1.5 metres, respectively.

$$\text{Mean depth} = D = \frac{2.0 + 1.9 + 1.8 + 1.6 + 1.5}{5} = \frac{8.8}{5} = 1.76 \text{ metres}$$

Values of deviations from the mean are (2.0 – 1.76), (1.9 – 1.76), (1.8 – 1.76), (1.6 – 1.76), (1.5 – 1.76) *i.e.*

0.24, 0.14, 0.04, – 0.16 and – 0.26.

The absolute values of these deviations from the mean, are 0.24, 0.14, 0.04, 0.16 and 0.26.

The average of these absolute values of deviations from the

$$\text{mean} = d = \frac{0.24 + 0.14 + 0.04 + 0.16 + 0.26}{5}$$

$$= \frac{0.84}{5} = 0.168 \text{ metre}$$

The water distribution efficiency

$$= \left(1 - \frac{d}{D}\right) = \left[1 - \frac{0.168}{1.76}\right] = 1 - 0.095 = 0.905$$

Hence, the water distribution efficiency = **0.905.** **Ans.**

Example 2.7. *A stream of 130 litres per second was diverted from a canal and 100 litres per second were delivered to the field. An area of 1.6 hectares was irrigated in 8 hours. The effective depth of root zone was 1.7 m. The runoff loss in the field was 420 cu. m. The depth of water penetration varied linearly from 1.7 m at the head end of the field to 1.1 m at the tail end. Available moisture holding capacity of the soil is 20 cm per metre depth of soil. It is required to determine the water conveyance efficiency, water application efficiency, water storage efficiency, and water distribution efficiency. Irrigation was started at a moisture extraction level of 50% of the available moisture.*

Solution.

(*i*) Water conveyance efficiency (η_c)

$$= \frac{\text{Water delivered to the fields}}{\text{Water supplied into the canal at the head}} \times 100$$

$$= \frac{100}{130} \times 100 = \mathbf{77\%} \text{ (say)} \quad \textbf{Ans.}$$

(*ii*) Water application efficiency (η_a)

$$= \frac{\text{Water stored in the root zone during irrigation}}{\text{Water delivered to the field}} \times 100$$

Water supplied to field during 8 hours @ 100 litres per second

$$= 100 \times 8 \times 60 \times 60 \text{ litres} = 2880 \text{ cu. m}$$

Runoff loss in the field = 420 cu. m.

$\therefore$ the water stored in the root zone

$$= 2880 - 420 = 2460 \text{ cu. m}$$

$\therefore$ Water application efficiency (η_a)

$$= \frac{2460}{2880} \times 100 = \mathbf{85.4\%} \quad \textbf{Ans.}$$

(*iii*) Water storage efficiency

$$= \frac{\text{Water stored in the root zone during irrigation}}{\text{Water needed in root zone prior to irrigation}} \times 100$$

Moisture holding capacity of soil

$$= 20 \text{ cm per m depth} \times 1.7 \text{ m depth of root zone} = 34 \text{ cm.}$$

Moisture already available in root zone at the time of start of irrigation

$$= \frac{50}{100} \times 34 = 17 \text{ cm.}$$

Additional water required in root zone

$$= 34 - 17 = 17 \text{ cm.}$$

$$= \frac{17}{100} \times (1.6 \times 10^4) \text{ cu. m} \qquad (i.e. \text{ Depth} \times \text{Plot area})$$

$$= 2720 \text{ cu. m}$$

But actual water stored in root zone = 2460 cu. m

∴ Water storage efficiency (η_s)

$$= \frac{2460}{2720} \times 100 = \mathbf{90\%} \quad \textbf{(say)} \quad \textbf{Ans.}$$

(*iv*) Water distribution efficiency

$$\eta_d = \left(1 - \frac{d}{D}\right)$$

where D = mean depth of water stored in the root zone

$$= \frac{1.7 + 1.1}{2} = 1.4 \text{ m.}$$

d is computed as below :

Deviation from the mean at upper end (absolute value)

$$= |1.7 - 1.4| = 0.3$$

Deviation from the mean at lower end

$$= |1.1 - 1.4| = 0.3.$$

d = Average of the absolute values of deviations from mean $= \frac{0.3 + 0.3}{2} = 0.30.$

Using Eq. (2.3), we have,

$$\eta_d = \left(1 - \frac{0.30}{1.4}\right) = \mathbf{0.786 \text{ or } 78.6\%} \quad \textbf{Ans.}$$

2.8. Consumptive Use or Evapotranspiration (C_u)

Consumptive use for a particular crop may be defined as the total amount of water used by the plant in transpiration (building of plant tissues, etc.) and evaporation from adjacent soils or from plant leaves, in any specified time. The values of consumptive use (C_u) may be different for different crops, and may be different for the same crop at different times and places.

In fact, the consumptive use for a given crop at a given place may vary throughout the day, throughout the month and throughout the crop period. Values of daily consumptive use or monthly consumptive use, are generally determined for a given crop and at a given place. Values of monthly consumptive use over the entire crop period, are then used to determine the irrigation requirement of the crop.

2.9. Effective Rainfall (R_e)

Precipitation falling during the growing period of a crop that is available to meet the evapo-transpiration needs of the crop, is called *effective rainfall.* It does not include precipitation lost through deep percolation below the root zone or the water lost as surface run off. Average ratios, applicable to effective rainfall, are shown in Table 2.4.

Table 2.4. Average Ratios Applicable to Effective Rainfall

Average annual rainfall in cm.	*Per cent chance of occurrence*				
	50	*60*	*70*	*80*	*90*
10	0.84	0.72	0.61	0.50	0.38
20	0.90	0.81	0.71	0.62	0.51
30	0.93	0.85	0.78	0.69	0.58
40	0.95	0.88	0.81	0.73	0.63
50	0.96	0.90	0.83	0.75	0.67
60	0.97	0.91	0.84	0.78	0.70
70	0.97	0.92	0.86	0.80	0.72
80	0.98	0.93	0.87	0.81	0.74
90	0.98	0.93	0.88	0.82	0.75
100	0.98	0.94	0.89	0.83	0.76
120	0.98	0.94	0.90	0.85	0.78
140	0.99	0.95	0.91	0.86	0.80
160	0.99	0.95	0.91	0.87	0.82
180	0.99	0.95	0.92	0.88	0.84
200	0.99	0.95	0.92	0.89	0.85

2.10. Consumptive Irrigation Requirement (CIR)

It is the amount of *Irrigation water* required in order to meet the evapotranspiration needs of the crop during its full growth. It is, therefore, nothing but the consumptive use itself, but exclusive of effective precipitation, stored soil moisture, or ground water. When the last two are ignored, then we can write

$$\text{C.I.R.} = C_u - R_e \qquad ...(2.4)$$

2.11. Net Irrigation Requirement (NIR)

It is the amount of irrigation water required in order to meet the evapotranspiration need of the crop as well as other needs such as leaching. Therefore, N.I.R. $= C_u - R_e +$ Water lost as percolation in satisfying other needs such as leaching.

2.12. Factors Affecting Consumptive Use

Consumptive use or evapotranspiration depends upon all those factors on which evaporation and transpiration depend ; such as temperature, sunlight, humidity, wind movement, etc, as detailed in article 7.34.2.3.

Example 2.8. *The following table gives the values of consumptive uses and effective rainfalls for the periods shown against them, for a Jowar crop sown at Bellary in Karnataka State. The period of growth is from 16th October to 2nd Feb., i.e. (110 days). Determine the net irrigation requirement of this crop, assuming that water is not required for any other purpose except that of fulfilling the evapotranspiration needs of the crop.*

Table 2.5 (*a*)

Dates (1)	C_u *in mm.* (2)	R_e *in mm.* (3)
October 16—31	37.0	30.8
November 1—30	84.2	20.4
December 1—31	154.9	6.7
January 1—31	188.1	2.4
February 1—2	13.3	1.0

Solution. The given table is extended as shown in Table 2.5 (*b*). Values in col. (4) are obtained by subtracting values of col. (3) from values of col. (2).

Table 2.5 (*b*)

Dates (1)	C_u (2)	R_e (3)	$NIR = C_u - R_e$ (4)
October 16—31	37.0	30.8	6.2
November 1—30	84.2	20.4	63.8
December 1—31	154.9	6.7	148.2
January 1—31	188.1	2.4	185.7
February 1—2	13.3	1.0	12.3
			$\Sigma = 416.2$ mm **= 41.62 cm**

Hence, the net irrigation requirement = **41.62 cm.** **Ans.**

2.13. Estimation of Consumptive Use

Although various methods have been developed in order to estimate evapotranspiration (consumptive use) values of different crop in an area, or for areas vegetated with the same cropping pattern, but the most simple and commonly used methods are :

(1) Blaney-Criddle Equation, and

(2) Hargreaves class *A* pan evaporation method.

(3) Penman's equation.

They are described below :

2.13.1. Blaney-Criddle Formula. It states that the monthly consumptive use is given by

$$C_u = \frac{k \cdot p}{40}\left[1.8t + 32\right] \qquad ...(2.5)$$

where, C_u = Monthly consumptive use in cm.

k = **Crop factor,** determined by experiments for each crop, under the environmental conditions of the particular area.

t = Mean monthly temperature in °C.

p = Monthly per cent of annual day light hours that occur during the period.

If $\frac{p}{40}\left[1.8t + 32\right]$ is represented by f, we get

$$C_u = k \cdot f \qquad ...(2.6)$$

This formula has been extensively used throughout the world for estimating seasonal water requirements. However, it was found that the values of k based on seasonal determinations were too low for the short periods between irrigations. This led to further developments and finally the formula was expressed as

$$C_u = k \Sigma f \qquad ...(2.7)$$

where C_u = Seasonal consumptive use, *i.e.* consumptive use during the period of growth for a given crop in a given area.

The above formula involves the use of crop factor, the value of which is to be determined for each crop and for different places. At present, this information is not available in India. Moreover, this formula does not take into consideration the factors such as humidity, wind velocity, elevation, etc. on which consumptive use depends. Hargreaves class A Pan evaporation method is, therefore, generally used in India.

Example 2.9. *Wheat is to be grown at a certain place, the useful climatological conditions of which are tabulated below in Table 2.6 (a). Determine the evapotranspiration and consumptive irrigation requirement of wheat crop. Also determine the field irrigation requirement if the water application efficiency is 80%. Make use of Blaney-Criddle equation and a crop factor equal to 0.8.*

Table 2.6 (*a*)

Month	*Monthly temp. in °C, averaged over the last 5 years*	*Monthly per cent of day time hr. of the year computed from the Sun-shine Tables*	*Useful rainfall in cm, averaged over the last 5 years*
(1)	(2)	(3)	(4)
November	18.0	7.20	1.7
December	15.0	7.15	1.42
January	13.5	7.30	3.01
February	14.5	7.10	2.25

Solution. Blaney-Criddle equation is

$$C_u = k \cdot \Sigma f \qquad \text{where, } f = \frac{p}{40}\left(1.8\,t + 32\right); \text{ and } k = 0.8 \text{ (given)}$$

Values of f are worked out in col. (5) of Table 2.6 (*b*).

Table 2.6 (*b*)

Month	t	p	R_e	$f = \frac{p}{40}(1.8t + 32)$
(1)	(2)	(3)	(4)	(5)
November	18.00	7.20	1.70	11.6
December	15.00	7.15	1.42	10.5
January	13.50	7.30	3.01	10.3
February	14.50	7.10	2.25	10.3
			$\Sigma = 8.38$ cm	$\Sigma = 42.7$ cm

$$C_u = k \cdot \Sigma f = 0.8 \times 42.7 = 34.16 \text{ cm.}$$

Hence, Consumptive use $C_u = 34.16$ cm. **Ans.**

Consumptive irrigation requirement = $C_u - R_e = 32.72 - 8.38 = 24.34$ cm. **Ans.**

Field irrigation requirement = C.I.R./$\eta_a = \frac{24.34}{0.8} = 30.43$ cm. **Ans.**

Example 2.10. *Determine the volume of water required to be diverted from the head works to irrigate area of 5000 ha using the data given in the table below. Assume 80% as the effective precipitation to take care of the consumptive use of the crop. Also assume 50% efficiency of water application in the field and 75% as the conveyance efficiency of canal*

Table 2.7 (*a*)

Month (1)	*Temp F* (2)	*Percentage hrs of sunshine* (3)	*Rainfall mm* (4)	*Consumptive coefficient or Crop factor (k)* (5)
June	70.8	9.90	75	0.80
July	74.4	10.20	108	0.85
August	72.8	9.60	130	0.85
September	71.6	8.40	115	0.85
October	69.3	7.86	105	0.65
November	55.2	7.25	25	0.65
December	47.1	6.42	0	0.60
January	48.8	8.62	0	0.60
February	53.9	9.95	0	0.65
March	60.0	8.84	0	0.70
April	62.5	8.86	0	0.70
May	67.4	9.84	0	0.75

Solution. The given table 2.7 (*a*) is extended as shown in table 2.7 (*b*) to compute monthly values of consumptive use (C_u) by the Eqn. $C_u = k \cdot f$, where $f = \frac{p}{40}[1.8t + 32]$, where t is in °C. When t is in °F as given in col (2) of Table 2.7 (*a*), then the eqn. becomes

$$C_u = k\left(\frac{p}{40}t\right) \quad \text{where } t \text{ is °F.}$$

Monthly values of f are hence worked out in col (6) of Table 2.7 (*b*).

Table 2.7 (*b*)

Month (1)	*Temp* °F (2)	*Percentage hrs of sun shine p* (3)	*Rainfall cm* (4)	*Consumptive coefficient or Crop factor k* (5)	$Cu = kf = \frac{k \cdot p}{40} \cdot t$ $\frac{\text{col(5)} \times \text{col (3)} \times \text{col (2)}}{40}$ cm. (6)
June	70.8	9.90	7.5	0.80	14.02
July	74.4	10.20	10.8	0.85	16.13
August	72.8	9.60	13.0	0.85	14.85
September	71.6	8.40	11.5	0.85	12.78
October	69.3	7.86	10.5	0.65	8.85
November	55.2	7.25	2.5	0.65	6.50
December	47.1	6.42	0	0.60	4.54
January	48.8	8.62	0	0.60	6.31
February	53.9	9.95	0	0.65	8.71
March	60.0	8.84	0	0.70	9.28
April	62.5	8.85	0	0.70	9.68
May	67.4	9.84	0	0.75	12.44
Σ			55.8		124.09 cm

Total consumptive use = Σ col(6) = 124.09 cm.

Useful rainfall = 80% of total precipitation (given)

= 80% × 55.8 cm. = 44.64 cm.

∴ Net irrigation requirement

$$NIR = C_u - R_e = 124.09 - 44.64 = 79.45 \text{ cm.}$$

Field irrigation requirement $= FIR = \dfrac{N \cdot I \cdot R}{\eta_a} = \dfrac{79.45}{50\%} = \dfrac{79.45}{0.5} = 158.9$ cm

Gross irrigation requirement at headworks $= \dfrac{FIR}{\eta_c}$

where η_c = conveyance efficiency = 75% = 0.75

∴ $$GIR = \frac{158.9}{0.75} = 211.87 \text{ cm.}$$

Vol. of water required for 5000 ha area

$$= \frac{211.87}{100} \text{ m} \times (5000 \times 10^4 \text{ m}^2) = \mathbf{105.93\ M{\cdot}m^3}. \quad \textbf{Ans.}$$

Example 2.11. *The monthly consumptive use values for Paddy are tabulated in Table 2.8. Determine the total consumptive use. What is the average monthly consumptive use and peak monthly consumptive use ?*

Table 2.8

Dates		*Rice (Loam Soil)* C_u *in cm*
June	1–30 } Nursery	26.69
July	1–12 }	8.76
July	13—31	14.38
August	1—31	22.73
September	1—30	21.29
October	1—31	25.50
November	1—24	15.06

Solution. The summation of consumptive uses

$$= 29.69 + 8.76 + 14.38 + 22.73 + 21.29 + 25.50 + 15.06 = 137.41 \text{ cm}$$

Hence, total consumptive use for paddy = 137.41 cm. **Ans.**

Average daily consumptive use

$$= \frac{137.41}{\text{Period of growth in days}} = \frac{137.41}{30 + 31 + 31 + 30 + 31 + 24}$$

$$= \frac{137.41}{177} = 0.77 \text{ cm.} = 7.7 \text{ mm.} \quad \textbf{Ans.}$$

Average monthly consumptive use = 0.77 × 30 = 23.1 mm. **Ans.**

Peak monthly consumptive use

= **26.69 cm.** (Highest value given) **Ans.**

2.13.2. Hargreaves Class A pan evaporation method. In this method, evapotranspiration (consumptive use) is related to pan evaporation by a constant K, called *consumptive use coefficient.* The formula can be written as

$$\frac{\text{Evapotranspiration}(E_t \text{ or } C_u)}{\text{Pan evaporation } (E_p)} = K$$

or

$$\boxed{E_t \text{ or } C_u = K \cdot E_p} \qquad ...(2.8)$$

Consumptive use coefficient (K) is different for different crops and is different for the same crop at different places. It also varies with the crop growth, and is different at different crop stages for the same crop. The above relationship is now available for various crops from many countries such as Israel, Philippines, U.S.A. and India. Research stations constantly go on reporting more and more data. Where specific data are not available, average values can be used as recommended by Hargreaves, and given in Table 2.9. The crops have been divided into 8 groups and the coefficients have been suggested for average conditions of soil, etc.

(*i*) *Group A.* The important crops include :

Sugar, Beats, Maize, Cotton, Jowar, Bean, Peas, Potatoes, etc.

(*ii*) *Group B.* This group consists of deciduous fruits and some field crops. Important crops are :

Tomatoes, Hybrid Walnuts, Plumes, Olives, and some group *A* crops that fail to produce maximum vegetative cover and maximum growth ratios.

(*iii*) *Group C.* $\frac{E_t}{E_p}$ ratios are of the order of 0.6. It includes crops like Melons, Onions, Carrots, Hops, Grapes, etc.

(*iv*) *Group D.* The maximum $\frac{E_t}{E_p}$ ratio is about 0.90 and usually occurs at about 75 to 80% completion of crop vegetative cycle. The important crops are :

Wheat, Barley, Celery, Flax and other small grains, etc.

(*v*) *Group E.* Ratios of $\frac{E_t}{E_p}$ vary from 0.7 to 1.10. The model value being 0.90. The important crops are :

Pastures, Orchard with cover crop, Plantain, etc.

(*vi*) *Group F.* It includes citrus crops such as Oranges, Grape fruit, etc. The $\frac{E_t}{E_p}$ ratios are fairly constant throughout the year and average to a value of about 0.60.

(*vi*) *Group G.* $\frac{E_t}{E_p}$ values generally increase with crop and vary from 0.66 to 1.00. It includes *Sugarcane and Alfalfa.*

(*viii*) *Paddy or Rice.* $\frac{E_t}{E_p}$ increases from 0.80 to 1.30, with crop growth and then falls down, reaching its maximum value somewhere near 50% growth, as shown in Table 2.9.

The coefficients shown in Table 2.9 are only average values and care must be taken while using them. Local values when available should only be used. The factors which increase or decrease the evapotranspiration may be taken into consideration. For ex-

ample, taller and more uneven vegetation tends to result in greater turbulence and more efficient utilisation of radiation in the production of water use. Dark green vegetation produces higher rate of absorption of solar energy, and hence, its evapotranspiration rate will be higher than that for light green vegetation. For this reason, the plant diseases, causing yellowing of the leaves of the plants, greatly reduce evapotranspiration.

Table 2.9. Hargreave's Average Values of Consumptive Use Coefficient K ($E_t = KE_p$)

Per cent of crop growing season	*Consumptive use coefficient (K) to be multiplied by class A Pan Evaporation (E_p), i.e. $E_t = K \cdot E_p$*							
	Group A	*Group B*	*Group C*	*Group D*	*Group E*	*Group F*	*Group G*	*Rice*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
0	0.20	0.15	0.12	0.08	0.90	0.60	0.50	0.80
5	0.20	0.15	0.12	0.08	0.90	0.60	0.55	0.90
10	0.36	0.27	0.22	0.15	0.90	0.60	0.60	0.95
15	0.50	0.38	0.30	0.19	0.90	0.60	0.65	1.00
20	0.64	0.48	0.38	0.27	0.90	0.60	0.70	1.05
25	0.75	0.56	0.45	0.33	0 90	0.60	0.75	1.10
30	0.84	0.63	0.50	0.40	0.90	0.60	0.80	1.14
35	0.92	0.69	0.55	0.46	0.90	0.60	0.85	1.17
40	0.97	0.73	0.58	0.52	0.90	0.60	0.90	1.21
45	0.99	0.74	0.60	0.58	0.90	0.60	0.95	1.25
50	1.00	0.75	0.60	0.65	0.90	0.60	1.00	1.30
55	1.00	0.75	0.60	0.71	0.90	0.60	1.00	1.30
60	0.99	0.74	0.60	0.77	0.90	0.60	1.00	1.30
65	0.96	0.72	0.58	0.82	0.90	0.60	0.95	1.25
70	0.91	0.68	0.55	0.88	0.90	0.60	0.90	1.20
75	0.85	0.64	0.51	0.90	0.90	0.60	0.85	1.15
80	0.75	0.56	0.45	0.90	0.90	0.60	0.80	1.10
85	0.60	0.45	0.36	0.80	0.90	0.60	0.75	1.00
90	0.46	0.35	0.28	0.70	0.90	0.60	0.70	0.90
95	0.28	0.21	0.17	0.60	0.90	0.60	0.55	0.80
100	0.20	0.20	0.17	0.60	0.90	0.60	0.50	0.20

Values of K for certain crops reported from India and U.S.A. are given in Tables 2.10 and 2.11 respectively.

Table 2.10. Values of K for Certain Indian Crops ; ($E_t = K \cdot E_p$)

% of crop growing season	*Wheat Ludhiana (India)*	*Wheat Poona (India)*	*Cotton Poona (India)*	*Maize Ludhiana (India)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
0	0.14	0.30	0.22	0.40
5	0.17	0.40	0.22	0.42
10	0.23	0.51	0.23	0.47
15	0.33	0.62	0.24	0.54
20	0.45	0.73	0.26	0.63
25	0.60	0.84	0.35	0.75
30	0.72	0.92	0.58	0.85
35	0.81	0.96	0.80	0.96
40	0.88	1.10	0.95	1.04
45	0.90	1.10	1.03	1.07
50	0.91	1.00	1.08	1.09
55	0.90	0.91	1.08	1.10
60	0.89	0.80	1.07	1.11
65	0.86	0.65	1.05	1.10
70	0.83	0.51	1.00	1.07
75	0.80	0.40	0.93	1.04
80	0.76	0.30	0.85	1.00
85	0.71	0.20	0.73	0.97
90	0.65	0.12	0.62	0.89
95	0.58	0.10	0.50	0.81
100	0.51	0.10	0.40	0.70
Seasonal value K	0.61	0.61	0.68	0.86

Table 2.11. Values of K for certain U.S.A. Crops ($E_t = K \cdot E_p$)

% of crop growing season	*Sugarcane* Hawaii (USA)*	*Rice Los Banos (USA)*	*Maize Albama (USA)*	*Jowar Albama (USA)*	*Sugarcane* Hargreaves*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0.34	1.00	0.40	0.42	0.48
5	0.37	1.02	0.40	0.44	0.50
10	0.40	1.03	0.43	0.46	0.53
15	0.44	1.05	0.46	0.48	0.55
20	0.50	1.07	0.52	0.50	0.60
25	0.60	1.09	0.59	0.52	0.67
30	0.72	1.11	0.67	0.56	0.75
35	0.86	1.13	0.76	0.59	0.80
40	0.93	1.16	0.85	0.64	0.85
45	0.98	1.18	0.93	0.71	0.87
50	1.02	1.20	1.02	0.79	0.89
55	1.05	1.21	1.09	0.92	0.90
60	1.07	1.22	1.14	1.01	0.90
65	1.10	1.22	1.19	1.07	0.85
70	1.13	1.21	1.21	1.09	0.75
75	1.16	1.19	1.23	1.09	0.71
80	1.19	1.16	1.22	1.05	0.68
85	1.20	1.10	1.16	0.99	0.59
90	1.20	1.03	1.06	0.91	0.51
95	1.19	0.96	0.95	0.82	0.50
100	1.19	0.86	0.75	0.70	0.50
Seasonal K	0.89	1.10	0.86	0.75	0.69

*Value of K for sugarcane reported from Hawaii and found by Hargreaves are compared in col. (2) and col. (6).

Class A pan evaporation (E_p) measurements. E_p can be experimentally determined by directly measuring the quantity of water evaporated from the standard class A pan. This pan is 1.2 m in diameter, 25 cm deep, and bottom is raised 15 cm above the ground surface. The depth of water is to be kept in a fixed range such that the water surface is at least 5 cm, and never more than 7.5 cm, below the top of the pan.

The pan evaporation (E_p) can also be determined by using the **Christiansen formula,** which states

$$E_p = 0.459\ R \cdot C_t \cdot C_w \cdot C_h \cdot C_s\, C_e \qquad \ldots(2.9)$$

where R = Extra-terrestrial radiation in the same units as E_p in cm or mm (Table 2.12),

C_t = Coefficient for temperature, and is given by

$$C_t = 0.393 + 0.02796\,T_c + 0.0001189\,T_c^2 \quad ...(2.10)$$

where, T_c is the mean temperature in °C.

C_w = Coefficient for wind velocity, and is given as

$$= 0.708 + 0.0034\,W - 0.0000038\,W^2 \quad ...(2.11)$$

where, W is the mean wind velocity at 0.5 metre above the ground in km/day.

C_h = Coefficient for relative humidity, and is given by

$$C_h = 1.250 - 0.0087H + 0.75 \times 10^4\,H^2 - 0.85 \times 10^{-8}\,H^4 \quad ...(2.12)$$

where H is the mean percentage relative humidity at noon or average relative humidity for 11 and 18 hours.

C_s = Coefficient for per cent of possible sunshine, and is given by

$$= 0.542 + 0.008S - 0.78 \times 10^{-4}\,S^2 + 0.62 \times 10^{-6}\,S^3 \quad ...(2.13)$$

where S is the mean sunshine percentage.

C_e = Coefficient of elevation

$$= 0.97 + 0.00984\,E \quad ...(2.14)$$

where E is the elevation in 100 metres.

Values of R for different latitudes are tabulated in Table 2.11.

Example 2.12. *(a) Determine the pan evaporation from the following data for the month of April, using Christiansen method.*

Latitude 15° 19′ N, Elevation + 449 metres

Month : April.

Mean Temperature 31.8°C.

Mean wind velocity at 0.5 m above the ground = 183 kilometres per day.

Mean relative humidity = 40%

Mean sunshine per cent = 89%

Use tables for extra-terrestrial radiation (Table 2.12). (b) What is the consumptive use for April in this country for a crop having a consumptive use coefficient equal to 0.80.

Solution. Find the value of R from Table 2.12 for the month of April and for a latitude of 15° 19′ N. It comes out to be about **47.3 cm**. Now, using eqs. (2.10) to (2.14), we get

$$C_t = 0.393 + 0.02796 \times 31.8 + 0.0001189\,(31.8)^2 = 1.403$$

Table 2.12. Mean Monthly Values of Extra-terrestrial Radiation R in cm.

Latitudes in degrees	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*
North												
45	15.621	19.990	31.953	41.072	50.317	52.146	52.426	46.101	35.204	25.730	16.891	13.843
40	19.609	23.393	35.027	42.926	50.003	52.146	52.730	47.498	37.770	29.312	20.701	17.831
35	23.546	26.670	37.871	44.450	51.384	51.918	52.730	48.565	40.056	32.664	24.460	21.819
30	27.407	29.794	40.411	45.669	51.460	51.384	52.451	49.276	42.037	35.814	28.092	25.781
25	31.140	32.690	42.647	46.558	51.206	50.571	51.841	49.682	43.688	38.710	31.572	29.642
20	34.722	35.382	44.552	47.117	50.597	49.428	50.902	49.708	45.009	41.300	34.900	33.376
15	38.100	37.821	46.126	47.320	49.657	47.980	49.657	49.403	45.989	43.612	37.973	36.906
10	41.250	40.005	47.346	47.168	48.388	46.253	48.082	48.717	46.609	45.593	40.818	40.259
5	44.120	41.885	48.209	46.660	46.787	44.221	46.178	47.701	46.888	47.244	43.409	43.358
0	46.736	43.485	48.692	45.822	44.882	41.910	43.993	46.335	46.812	48.539	45.695	46.177

$$C_w = 0.708 + 0.0034 \times 183 - 0.0000038\,(183)^2 = 1.200.$$

$$C_h = 1.250 - 0.0087 \times 40 + 0.75 \times 10^4\,(40)^2 - 0.85 \times 10^{-8}\,(40)^4 = 1.000$$

$$C_s = 0.542 + 0.008 \times 89 - 0.78 \times 10^{-4}(89)^2 + 0.62 \times 10^{-6}(89)^3 = 1.073.$$

$$C_e = 0.97 + 0.00984 \times 4.49 = 1.014.$$

(*a*) Pan evaporation E_p is given by eq. (2.9) as :

$$E_p = 0.459\,R \cdot C_t \cdot C_w \cdot C_h \cdot C_s \cdot C_e$$
$$= 0.459\,(47.3)\,(1.403)\,(1.200)\,(1.000)\,(1.073)\,(1.014)$$
$$= 39.8 \text{ cm.} \quad \textbf{Ans.}$$

(*b*) $E_t = K \cdot E_p = 0.8 \times 39.8 = 31.84$ cm

Hence, the required value of consumptive use = 31.84 cm. **Ans.**

Example 2.13. *Determine the consumptive use and net irrigation requirement for Jowar sown at Bellary (Karnataka) from the following data:*

Table 2.13 (*a*)

Dates and period of growth	*Pan evaporation E_p or P_e in cm*	*Consumptive use coefficient (K)*	*Effective precipitation in cm.*
(1)	*(2)*	*(3)*	*(4)*
Oct. 16—31	8.49	0.44	3.42
Nov. 1—30	15.57	0.54	2.19
Dec. 1—31	16.59	0.94	0.54
Jan. 1—31	19.10	0.99	0.15
Feb. 1—2	1.54	0.73	0.02

Solution.

Proceed as shown in Table 2.13 (*b*).

Table 2.13 (*b*)

Dates	*E_p (cm)*	*K*	*$K \cdot E_p = E_t = C_u$ in cm*	*R_e in cm*	*$C_u - R_e$ = N.I.R.*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
Oct. 16—31	8.49	0.44	3.74	3.42	0.32
Nov. 1—30	15.57	0.54	8.41	2.19	6.22
Dec. 1—31	16.59	0.94	15.59	0.54	15.05
Jan. 1—31	19.10	0.99	18.91	0.15	18.76
Feb. 1—2	1.54	0.73	1.12	0.02	1.10
			$\Sigma = 47.77$		$\Sigma = 41.45$

First of all, calculate the values of C_u as in col. (4) Table 2.13 (*b*) and then determine N.I.R. as in col. (6) of this Table. Table is otherwise self-explanatory.

Total consumptive use = **47.77 cm**
Net Irrigation requirement = **41.45 cm** } **Ans.**

Table 2.14. Sample Calculations for Determining Irrigation Requirement of Cotton Crop. (in cm)

Cotton (Oct. 11—Feb. 26 = 139 days)

Dates	*No. of days up to mid-point of interval*	*% of growing season = col. (2) × 100/139*	*Pan Evap. = E_p*	*K*	*$E_pK = C_u$*	*Effective rainfall R_e*	*N.I.R. in cm*	*F.I.R. = N.I.R./η_a*	*GIR = FIR/η_c*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
Oct. 11—31	11	7.9	14.73	1.23	3.38	—	3.38	3.96	5.28
Nov. 1—30	35	25.2	18.85	0.36	6.78	—	6.78	7.98	10.65
Dec. 1—31	67	48.2	17.60	1.00	17.60	—	17.60	20.70	27.60
Jan. 1—31	98	70.5	19.63	0.99	19.43	—	19.43	22.86	30.48
Feb. 1—26	126	90.6	19.94	0.61	12.17	—	12.17	14.30	18.07
					$\Sigma = 59.36$		$\Sigma = 59.36$	$\Sigma = 69.80$	$\Sigma = 92.08$

FIR = Field Irrigation Requirement (in cm) $= \frac{NIR}{\eta_a} = \frac{NIR}{0.85}$

FIR includes percolation losses in the field water-courses, field channels and in field application of water.

GIR = Gross Irrigation Requirement (in cm) $= \frac{FIR}{\eta_c} = \frac{FIR}{0.85}$

i.e. GIR = FIR + Conveyance losses in distributaries up to the field.

Table 2.15

Sample Calculations for Determining Irrigation Requirement for Wheat Crop (in cm)

Wheat : Period of growth : 1 Nov. to 15 March (135 days)

Dates	*No. of days up to midpoint of interval*	*% of growing season = col. (2) × 100/135*	*Pan evaporation (E_p) in cm*	*Coeff. (K)*	*$E_t = K \cdot E_p = C_u$ Consumptive use in cm*	*R_e in cm*	*N.I.R. = $C_u - R_e$ cm*	*F.I.R. = NIR/η_a = NIR/0.7*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
Nov. 1—30	15	11	15	0.16	2.40	0.4	2.00	2.86
Dec. 1—31	46	34	12.8	0.46	5.89	1.6	4.29	6.13
Jan. 1—31	77	57	9.4	0.73	6.86	3.2	3.66	5.23
Feb. 1—28	106	78	15.4	0.90	13.86	2.2	11.66	16.66
March 1—15	128	95	10.1	0.60	6.06	Nil	6.06	8.66
					Σ = 35.07		Σ = 27.67	Σ = 39.54

Note : 1. The peak requirement during a month will be more than the average ; allowance should therefore, be made for this, while designing channel capacity.

2. Effective rainfall (R_e) should be taken from the previous records, and the mean value should not be taken. On the other hand, the values which are available in 75% to 80% of the years should be taken.

2.13.3. Penman's Equation. While the *Blaney Criddle equation* (1975) and the *Hargreaves class A pan equation using Christiansen formula* (1968) had been in use for the last many years for computing the consumptive use, C_u (*i.e.* evapotranspiration, E_t) values, and net irrigation requirements for different crops ; the *Penman equation* (1998) has, however, more recently been introduced for determing the consumptive use of different areas or different segments of a basin, depending upon the type of vegetation covering each sub-basin. The advantage with this equation lies in the fact that the different specified values of *coefficient of reflection* (*albedo*), a factor used in this equation, are available for different types of areas, which can be used in Penman's equation to compute consumptive use (*i.e.* Potential evapotranspiration, PAT) values for different segments of command area.

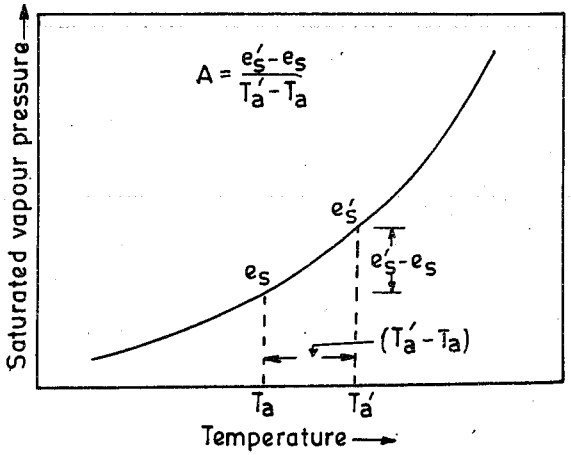


Fig. 2.3. Saturation vapour pressure *vs.* Temp. curve.

Penman's equation for computation of *PET* or C_u for an area, has a sound theoretical reasoning, and it is not a simple empirical equation. This equation has, infact, been derived by intelligently combining the *energy balance* and *mass transfer approaches* of the computations of transpiration and evaporation, respectively. Hence, although slightly complicated mathematical conceptual work is involved here, yet its use is becoming more and more popular, in today's modern computer age.

Penman's equation, incorporating some of the modifications suggested by other investigators, is given as:

$$E_t = \frac{A \cdot H_n + E_a \cdot \gamma}{A + \gamma} \qquad ...(2.15)$$

where E_t = Daily potential evapo-transpiration

A = Slope of the saturation vapour pressure V_s Temp. curve at the mean air temperature, as shown in Fig. 2.3, and values given in Table 2.16.

H_n = Net incoming solar radiation or energy, expressed *in mm of evaporable water per day*[*]

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* Since the energy required by water in evaporation equals 585 calories/gm = 585 calories/cc (1 gm = 1 cc), we have

Energy as 1 mm of evaporable water from an area of A hectares

$$= \left(\frac{1}{10}\text{ cm}\right)(A \times 10^8\text{ cm}^2) \times 585\text{ cal/cm}^3$$

$$= \mathbf{585 \times 10^7\ A\ calories}$$

E_a = A parameter including wind velocity and saturation deficit, as given by Eqn. (2.18) in mm/day

γ = psychromatic constant
= 0.49 mm of Hg/°C

The net radiation (H_n) in the above equation is the same, as used in the energy budget equation (7.46), and is estimated by the equation

$$H_n = H_c\,(1-r)\left(a+b\cdot\frac{n}{N}\right) - \sigma\cdot T_a^4\,(0.56-0.092\sqrt{e_a})\times\left(0.10+0.90\,\frac{n}{N}\right) \quad ...(2.16)$$

where H_c = mean incident solar radiation at the top of the atmosphere on a horizontal surface, expressed in mm of evaporable water per day. This value is a function of latitude (ϕ) of the place and the period of the year, as per the mean monthly values given in table 2.18.

r = **reflection coefficient** (*albedo*) of the given area. Usual values of this coefficient for different types of areas are given in Table 2.17.

a = a constant depending upon the latitude (ϕ) and is given as

$$a = 0.29\cos\phi \quad ...(2.17)$$

b = a constant having average value
= 0.52

n = actual duration of bright sunshine in hours

N = maximum possible hours of bright sunshine (mean value). This value is a function of latitude (ϕ), and its values are given in table 2.19 for each month of the year

σ = *Stefan-Bolzman constant*
= 2.01×10^{-9} mm/day

T_a = mean air temperature in °K
= 273 + °C

e_a = actual mean vapour pressure in the air in mm of Hg.

The parameter E_a of Penman's equation (2.15) is estimated as :

$$E_a = 0.35\left(1+\frac{V_2}{160}\right)(e_s-e_a)\ \text{mm/day} \quad ...(2.18)$$

where V_2 = mean wind speed at 2 m above the ground in km/day

e_s = saturation vapour pressure at mean air temperature in mm of Hg (Table 2.16)

e_a = actual mean vapour pressure of air in mm of Hg.

With the help of the above equation, and using the values of A, e, r, H_c and N from tables 2.16 to 2.19, E_t or C_u can be determined for the given area. This equation can also be used to compute the evaporation from a water surface (lake, etc.) by using $r = 0.05$. Due to its general applicability, this equation is widely used these days in India, the U.K., the Australia, and in some parts of U.S.A.

Table 2.16. Saturation Vapour Pressure (e_s), and Slope of Saturation Vapour Pressure *Vs* Temperature Curve (A)

Temperature (1)	*Saturation vapour pressure (e_s) in mm of Hg* (2)	*Slope A in mm/°C* (3)
0	4.58	0.30
5.5	6.54	0.45
7.5	7.78	0.54
10.0	9.21	0.60
12.5	10.87	0.71
15.0	12.79	0.80
17.5	15.00	0.95
20.0	17.54	1.05
22.5	20.44	1.24
25.0	23.76	1.40
27.5	27.54	1.61
30.0	31.82	1.85
32.5	36.68	2.07
35.0	42.81	2.35
37.5	48.36	2.62
40.0	55.32	2.95
45.0	71.20	3.66

Table 2.17. Values of Reflection Coefficient r (albedo)

Surface	*Range of r values*
Close grained crops	0.15 – 0.25
Bare lands	0.05 – 0.45
Water surface	0.05
Snow	0.45 – 0.90

Table 2.18. Mean Monthly Solar Radiation at Top of Atmosphere H_c in mm of evaporable water/day

North Latitude	*Jan.*	*Feb.*	*March*	*April*	*May*	*Jun.*	*Jul.*	*Aug.*	*Sep.*	*Oct.*	*Nov.*	*Dec.*
0°	14.5	15.0	15.2	14.7	13.9	13.4	13.5	14.2	14.9	15.0	14.6	14.3
10°	12.8	13.9	14.8	15.2	15.0	14.8	14.8	15.0	14.9	14.1	13.1	12.4
20°	10.8	12.3	13.9	15.2	15.7	15.8	15.7	15.3	14.4	12.9	11.2	10.3
30°	8.5	10.5	12.7	14.8	16.0	16.5	16.2	15.3	13.5	11.3	9.1	7.9
40°	6.0	8.3	11.0	13.9	15.9	16.7	16.3	14.8	12.2	9.3	6.7	5.4
50°	3.6	5.9	9.1	12.7	15.4	16.7	16.1	13.9	10.5	7.1	4.3	3.0

Table 2.19. Mean Monthly Values of Possible Sunshine Hours (N)

North Latitude	Jan.	Feb.	March	April	May	Jun.	July	Aug.	Sep.	Oct.	Nov.	Dec.
0°	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1	12.1
10°	11.6	11.8	12.1	12.4	12.6	12.7	12.6	12.4	12.9	11.9	11.7	11.5
20°	11.1	11.5	12.0	12.6	13.1	13.3	13.2	12.8	12.3	11.7	11.2	10.9
30°	10.4	11.1	12.0	12.9	13.7	14.1	13.9	13.2	12.4	11.5	10.6	10.2
40°	9.6	10.7	11.9	13.2	14.4	15.0	14.7	13.8	12.5	11.2	10.0	9.4
50°	8.6	10.1	11.8	13.8	15.4	16.4	16.0	14.5	12.7	10.8	9.1	8.1

Example 2.14. *Compute the total consumptive use (C_u) from a drainage basin located near Gurgaon (Haryana) during the month of April by Penman's formula. The following data is given:*

Latitude of place = *28° N*
Elevation = *RL 220 m.*

Meteorologically observed data during April

Mean monthly temperature = *40°C*
Mean relative humidity = *35%*
Mean observed sun shine hours per day = *13h*
Mean wind velocity at 2 m height = *72km/day/(3km/h)*

Data available and obtained from standard charts

(i) Slope of the e_s Vs T° chart at 40°C = *2.95 mm of Hg/ °C*
(ii) Saturation pressure at 40°C = *55.32 mm of Hg*
Mean monthly solar radiation at top of atmosphere during April for 28°N latitude = *14.9 mm of evaporable water/day*
Mean monthly value of possible sunshine hours for April for 28°N latitude = *12.9 h*
Albedo for the area = *0.25*

Solution. Penman's equation (2.15) is given as:

$$E_t = C_u = \frac{A\,H_n + E_a \cdot \gamma}{A + \gamma}$$

where $A = 2.95$ mm of Hg/°C

H_n = to be computed by eq. (2.16)

E_a = given by eq. (2.18) as :

$$= 0.35\left(1 + \frac{V_2}{160}\right)(e_s - e_a) \text{ mm/day}$$

$$= 0.35\left(1 + \frac{72}{160}\right) \times (55.32 - 19.36)$$

$(\because\ e_a = (\text{R.H.})\ e_s$

$= 35\% \times 55.32 = 19.36$ mm Hg

$= 18.07$ mm/day

$\gamma = 0.49$ mm of Hg/°C

H_n is given by eqn. (2.16) as :

$$H_n = H_c (1-r)\left(a + b \cdot \frac{n}{N}\right) - \sigma \cdot T_a^4 (0.56 - 0.092\sqrt{e_a}) \times \left(0.10 + 0.90\frac{n}{N}\right)$$

where H_c = mean monthly incident solar radiation at top of atmosphere
= 14.9 mm of evaporable water/day
$r = 0.25$
$a = 0.29 \cos \phi = 0.29 \cos 28° = 0.256$
$b = 0.52$
$n = 13$ h
$N = 12.9$ h
$\sigma = 2.01 \times 10^{-9}$ mm/day
$T_a = (40°C + 273) = 313°K$
e_a = (R.H.) $e_s = 35\% \times 55.32$
= 19.36 mm of Hg

Substituting values, we get

$$H_n = 14.9\,(1-0.25)\left[0.256 + 0.52 \times \frac{13}{12.9}\right] -$$
$$\left[\{2.01 \times 10^{-9} \times (313)^4\} \times \{0.56 - 0.092\sqrt{19.36}\} \times \left\{0.10 + 0.90 \times \frac{13}{12.9}\right\}\right]$$
$$= 14.9\,(0.75)\,(0.78) - (19.292)\,(0.155)\,(1.007)$$
$$= 8.716 - 3.011$$
$$= \mathbf{5.705} \text{ mm of evaporative water/day}$$

Now, $$C_u = \frac{AH_n + E_a\gamma}{A + \gamma}$$
$$= \frac{2.95 \times 5.705 + 18.07 \times 0.49}{2.95 + 0.49}$$
$$= \frac{49.648 + 8.854}{3.44} = \mathbf{17.01\ mm/day.} \quad \textbf{Ans.}$$

2.13.4. Comparison of Blaney-Criddle Equation, Hargreaves-Christiansen Equation and Penman's Equation. These three empirical equations have been developed by the various researchers over the last 40 years to estimate evapo-transpiration (E_t) values for different crops, or area segments vegetated with the same cropping pattern, under different climatic variables. Since the suggested empirical equations are often subjected to rigorous local calibrations, they can not have a global validity. It, however, becomes difficult to suggest as to which equation should be used in a particular case. A recent study made in Chandigarh region has, however, shown that the annual evapotranspiration values obtained from Penman's equation are quite close to the values obtained from the actual field observations made in pan evaporation method ; while the values obtained by Blaney-Criddle equation were on much higher side (about 30% higher), and the values obtained by Hargreaves-Christiansen equation were on lower side (about 15-20% lower). No definite conclusion can, hence, be drawn regarding the decision on the global use of a particular equation.

Although, use of Penman's equation is being largely advocated these days, yet since the equation needs elaborate data, it may not be always feasible to use this equation. Moreover, this equation can be used for generalised vegetated areas, and not for individual crops, since the value range of reflection coefficient *i.e. albedo* (r), as used in this equation, is given for areas having close grained crops, as to vary between 0.15-0.25 (Pl. see table 2.17)

2.14. Soil-Moisture-Irrigation Relationship

The water below the watertable is known as ground water and above the watertable as soil-moisture.

Extending down from the ground surface, is the soil zone or the root zone, which is defined as being the depth of overburden that is penetrated by the roots of vegetation, as shown in Fig. 2.4. This zone is the most important from irrigation point of view, because it is this zone, from which the plants do take their water supplies. When water falls over the ground, a part of it gets absorbed in this root zone, and the rest flows downward under the action of gravity and is called *gravity water.*

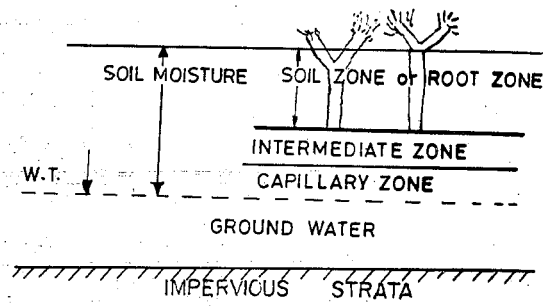


Fig. 2.4.

2.14.1. Field Capacity. Immediately after a rain or irrigation water application, when all the gravity water has drained down to the watertable, a certain amount of water is retained on the surfaces of soil grains by molecular attraction and by loose chemical bonds (*i.e.* adsorption). This water cannot be easily drained under the action of gravity, and is called the *field capacity.* The field capacity is thus the water content of a soil after free drainage has taken place for a sufficient period. This period of free gravity drainage is generally taken as 2 to 5 days.

The field capacity water further consists of two parts. One part is that which is attached to the soil molecules by surface tension against gravitation forces, and can be extracted by plants by *capillarity.* This water is called **capillary water.** The other part is that which is attached to the soil molecules by loose chemical bonds. This water which cannot be removed by capillarity is not available to the plants, and is called the **hygroscopic water.**

The field capacity water (*i.e.* the quantity of water which any soil can retain indefinitely against gravity) is expressed as the ratio of the weight of water contained in the soil to the weight of the dry soil retaining that water : *i.e.*

$$\text{Field Capacity} = \frac{\text{Wt. of water retained in a certain vol. of soil}}{\text{Wt. of the same volume of dry soil}} \times 100 \qquad ...(2.19)$$

If we consider 1 m^2 area of soil and d metre depth of root zone, then the volume of soil is $d \times 1 = d$ cubic metres. If the dry unit wt. of soil is γ_d kN/m^3*, then the Wt. of d cubic metres of soil is $\gamma_d \cdot d$ kN. If F is the field capacity, then

* It is the unit wt. of the dried soil sample and not of the soil solids. It may sometimes hence be called as **apparent unit wt.**

$$F = \frac{\text{Wt. of water retained in unit area of soil}}{\gamma_d d}$$

or Wt. of water retained in unit area of soil $= \gamma_d \cdot d \cdot F \text{ kN/m}^2$

$$\therefore \text{ Vol. of water stored in unit area of soil} = \frac{\gamma_d \cdot d \cdot F \text{ kN/m}^2}{\gamma_w \text{ kN/m}^3}$$

or Total water storage capacity of soil in (m depth of water)

$$= \frac{\gamma_d \cdot d \cdot F}{\gamma_w} \text{ m}$$

where F = the field capacity m.c.
d = depth of root zone in m
γ_w = the unit wt. of water
γ_d = the dry unit wt. of soil.

Hence, the depth of water stored in the root zone in filling the soil upto field capacity

$$= \frac{\gamma_d \cdot d \cdot F}{\gamma_w} \textbf{ metres.} \qquad ...(2.20)$$

The knowledge of field capacity is very important, because it is the field capacity water which can supply water for plant nourishment. The larger part of applied water drains down and joins the watertable and is thus a waste from irrigation point of view.

As expressed earlier, the total field capacity water cannot be utilised by the plants. The plants can extract water from the soil till the permanent wilting point is reached. The **permanent wilting point** is that water content at which plant can no longer extract sufficient water for its growth, and wilts up. It is the point at which permanent wilting of plants take place. *It, therefore, becomes evident that the water which is available to the plants, is the difference of field capacity water and permanent wilting point water. This is known as available moisture or maximum storage capacity of soil. Hence, the available water or* **available moisture** may be defined as the difference in water content of the soil between field capacity and permanent wilting point. The water left in the soil after the permanent wilting point is reached, cannot be removed, and is known as, unavailable moisture or *Hygroscopic water* (See Fig. 2.5).

2.14.2. Readily available moisture. It is that portion of the available moisture which is most easily extracted by the plants, and is approximately 75 to 80% of the available moisture.

2.14.3. Soil-moisture deficiency. The water required to bring the soil moisture content of a given soil to its field capacity is called the *field moisture deficiency* or *soil-moisture deficiency.*

2.14.4. Equivalent moisture. Just as the field capacity is the water retained by a saturated soil after being acted upon by gravity ; similarly, *equivalent moisture* is the water retained by a saturated soil after being centrifuged for 30 minutes by a centrifugal force of 1000 times that of gravity. Therefore, it is slightly less, or at the most equal to the field capacity.

2.15. Estimating Depth and Frequency of Irrigation on the Basis of Soil Moisture Regime Concept

Water or soil moisture is consumed by plants through their roots. It, therefore, becomes necessary that sufficient moisture remains available in the soil from the surface to the root zone depth. As explained earlier, the soil moisture in the root zone can vary between field capacity (upper limit) and wilting point moisture content (lower limit) as shown in Fig. 2.5.

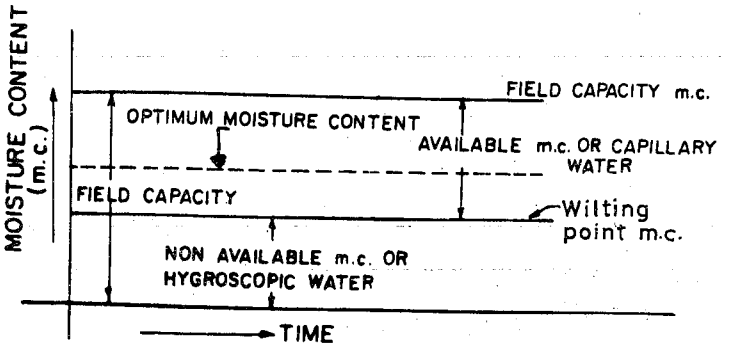


Fig. 2.5.

It is also evident from the previous discussion that the soil moisture is not allowed to be depleted up to the wilting point, as it would result in considerable fall in crop yield. The optimum level up to which the soil moisture may be allowed to be depleted in the root zone without fall in crop yield, has to be worked out for every crop and soil, by experimentation. *The irrigation water should be supplied as soon as the moisture falls up to this optimum level (fixing irrigation frequency) and its quantity should be just sufficient to bring the moisture content up to its field capacity, making allowance for application losses (thus fixing water depth).*

Water will be utilised by the plants after the fresh irrigation dose is given, and soil moisture will start falling. It will again be recouped by a fresh dose of irrigation, as soon as the soil moisture reaches the optimum level, as shown in Fig. 2.6.

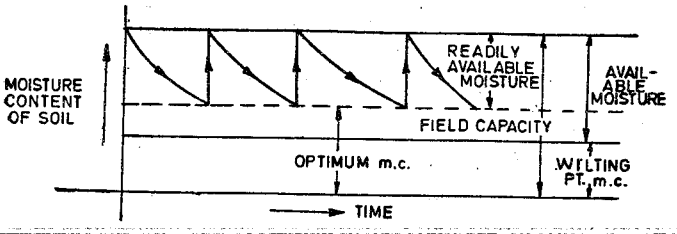


Fig. 2.6.

Example 2.15. *After how many days will you supply water to soil in order to ensure sufficient irrigation of the given crop, if*

(i) Field capacity of the soil = 28%

(ii) Permanent wilting point = 13%

(iii) Dry density of soil = 1.3 gm/c.c.

(iv) Effective depth of root zone = 70 cm

(v) Daily consumptive use of water for the given crop = 12 mm.

Assume any other data, not given. *(Engineering Services, 1974)*

Solution. We know, by definition of available moisture, that

the available moisture = Field capacity– Permanent wilting

$= 28 - 13 = 15\%.$

Let us assume that the readily available moisture or the optimum soil moisture level is 80% of available moisture.

i.e., Readily available moisture $= 0.80 \times 15\% = 12\%$

$\therefore$ Optimum moisture $= 28 - 12 = 16\%$

It means that the moisture will be filled by irrigation between 16% and 28%.

Depth of water stored in root zone between these two limits

$$= \frac{\gamma_d \cdot d}{\gamma_w} \left[\text{Field capacity m.c.} - \text{Optimum m.c.}\right]$$

where $\dfrac{\gamma_d}{\gamma_w} = \dfrac{\rho_d \cdot g}{\rho_w \cdot g} = \dfrac{\rho_d}{\rho_w} = \dfrac{1.3 \text{ gm/cc}}{1.0 \text{ gm/cc}} = 1.3$

$d = 0.7$ m (given)

$= 1.3 \times 0.70 \left[0.28 - 0.16\right]$ m

$= 1.3 \times 0.7 \times 0.12 \text{ m} = 0.1092 \text{ m} = 10.92 \text{ cm}.$

Hence, water available for evapo-transpiration = 10.92 cm.

1.2 cm of water is utilised by the plant in 1 day

$\therefore$ 10.92 cm of water will be utilised by the plant in

$$= \frac{1 \times 10.92}{1.2} \text{ days} = \mathbf{9.1 \text{ days}} \text{ ; Say 9 days.}$$

Hence, after 9 days, water should be supplied to the given crop. **Ans.**

Example 2.16. *Wheat is to be grown in a field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of the soil, if the dry unit weight of the soil is 14.72 kN/m^3. If irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to the field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water-courses and the field channels is 15% of the outlet discharge ?*

Solution. Maximum storage capacity or Available moisture.

$$= \frac{\gamma_d \cdot d}{\gamma_w} \left[\frac{\text{Field capacity m.c.}}{100} - \frac{\text{Wilting pt. m.c.}}{100}\right]$$

where $\gamma_d = 14.72$ kN/m^3

$d =$ depth of root zone $= 0.8$ m

$\therefore$ Max. storage capacity or max. Available moisture

$$= \frac{14.72}{9.81} \times 0.8 \left[0.27 - 0.13\right] \qquad \left[\because \gamma_w = 9.81 \text{ kN/m}^3\right]$$

$= 1.2\ [0.14] = 0.168$ metres $= 16.8$ cm. **Ans.**

Since the moisture is allowed to vary between 27% and 18%, the deficiency created in this fall

$$= \frac{14.72}{9.81} \times 0.8\,[0.27 - 0.18]$$

$$= 1.2 \times 0.09 = 0.108 \text{ metres} = \mathbf{10.8\ cm.}$$

Hence, 10.8 cm depth of water is the net irrigation requirement.

Quantity of water required to be supplied to the field (F.I.R.) $= \frac{\text{N.I.R.}}{\eta_a}$

or $$\text{F.I.R.} = \frac{\text{N.I.R.}}{0.80} = \frac{10.8}{0.80} = 13.5 \text{ cm.} \quad \textbf{Ans.}$$

Quantity of water needed at the canal outlet

$$= \frac{\text{F.I.R.}}{\eta_c} = \frac{13.5}{0.85} = 15.88 \text{ cm.} \quad \textbf{Ans.}$$

Example 2.17. *800 m^3 of water is applied to a farmer's rice field of 0.6 hectares. When the moisture content in the soil falls to 40% of the available water between the field capacity (36%) of soil and permanent wilting point (15%) of the soil crop combination, determine the field application efficiency. The root zone depth of rice is 60 cm. Assume porosity = 0.4.* *(Civil Services, 1994)*

Solution. We have defined Field Capacity m.c. (F) as :

$$F = \frac{\text{Wt. of water contained in a certain vol. of soil}}{\text{Wt. of the same volume of dry soil (}i.e.\text{ wt. of dry soil retaining that water)}}$$

If a saturated soil contains volume equal to V, and the volume of its voids is V_v, then the weight of water contained in this soil $= \gamma_w \cdot V_v$; where γ_w is the unit wt. of water. The wt. of this soil of V m^3 after it is oven dried to remove water and to fill the voids with air, is given by $\gamma_d \cdot V$; where γ_d is the *dry unit wt. of the soil.*

$$\therefore \quad F = \frac{\gamma_w \cdot V_v}{\gamma_d \cdot V} \qquad \text{But } \frac{V_v}{V} = n \text{ (porosity)}$$

$$\therefore \quad F = \frac{\gamma_w}{\gamma_d} \cdot n$$

$$\therefore \quad \frac{\gamma_d}{\gamma_w} = \frac{n}{F} = \frac{0.4}{0.36} = 1.11 \qquad \because \ n = \text{Porosity} = 0.4 \text{ (given)},\ F = \text{F.C.} = 0.36$$

Max. quantity of water stored between field capacity (FC) and permanent wilting point (P.W)

$$= \left(\frac{\gamma_d}{\gamma_w}\right) d \cdot (w_F - w_P)$$

where d = root zone depth = 0.6 m (given)

$$= 1.11 \times 0.60\,[0.36 - 0.15] = 0.14 \text{ m.}$$

Deficiency of water created when irrigation is done

$$= 60\% \times 0.14 \text{ m} \left[\because \text{ irrigation water is applied when m.c. falls to 40\% of m.c. available between F.C. and P.W.}\right]$$

$$= \mathbf{0.084\ m}$$

Hence, irrigation water is supplied to fill up 0.084 m depth of water.

$\therefore$ Vol. of irrigation water required to fill up the created deficiency

$= 0.084\text{ m} \times (0.6\text{ hect.})$

$= 0.084\text{ m} \times (0.6 \times 10{,}000)\text{ m}^2 = \mathbf{504\ m^3}$. ...(*i*)

Actual irrigation water supplied $= 800\text{ m}^3$ ...(*ii*)

$\therefore$ Efficiency of field application $= \dfrac{504}{800} = \mathbf{63\%}$ **Ans.**

Example 2.18. *Work out the irrigation schedule based on the soil moisture concept, given the following information. Also extract the data on the total depth of irrigation water required and the respective dates of irrigation water supply :*

(a) The crop is grown in an appropriate soil with no restrictive layers within the top 1.5 m depth of soil.

(b) Normal root zone depth of the crop is 1.2 m.

(c) Bulk density of soil is 1.35.

(d) Field capacity is 18% and permanent wilting point is 7%.

(e) Moisture level in the soil is to be maintained at not less than one-third of available retention. Irrigation will then be done over a duration of 2 days at a uniform rate of supply and at a uniform rate of advance to fully and just compensate for the depletion.

(f) No extra water is ever required for leaching.

(g) Sowing is done on 1 November when the soil moisture is left just at field capacity in the entire root zone.

(h) For the crop, at the location, the average evapotranspiration rates are :

1 Nov. — 30 Nov.	:	*1.1 mm/day*
1 Dec. — 31 Dec.	:	*1.7 mm/day*
1 Jan. — 31 Jan.	:	*2.4 mm/day*
1 Feb. — 28 Feb.	:	*1.5 mm/day*
1 March—25 March	:	*3.5 mm/day*

(i) Harvesting is done on or after 26 March.

(j) There is expected an effective rainfall of 24 mm during 4 January to 19 January, both days inclusive, with uniform intensity.

(k) By the end of the crop growth season, only the minimum water needed to be left unused in the root zone. *(Engineering Services, 1990)*

Solution.

Max. moisture retained by soil = Field capacity = 18%

Permanent wilting *i.e.* below which soil cannot extract water for plant's growth

$= 7\%$

$\therefore$ Max. moisture available for plant's growth *i.e. available moisture retention*

$= 18 - 7 = 11\%$

$\frac{1}{3}$ of available moisture $= \dfrac{11}{3}\% = 3.67\%$

$\therefore$ Moisture level at which irrigation must start = Minimum m.c. at which plants start wilting $+ \frac{1}{3}$ of available moisture (given)

$$= 7\% + 3.67\% = 10.67\%.$$

This means that we will start irrigation as soon as m.c. falls to 10.67%, and will, thus, fill the soil with moisture till it rises to 18% (field capacity).

Irrigation water required to increase m.c. of soil in root zone from 10.67% to 18% is obtained by equation (2.15), as :

$$= \frac{\gamma d}{w}\left[\begin{matrix}\text{Upper limit m.c.} \\ \text{as fraction}\end{matrix} - \begin{matrix}\text{Lower limit m.c.} \\ \text{as fraction}\end{matrix}\right]$$

where γ = Unit wt. (Apparent) of soil

w = Unit wt. of water

$\therefore$ $\frac{\gamma}{w}$ = Density (apparent) of soil = 1.35 (given)

d = root zone depth = 1.2 m (given)

$$= 1.35 \times 1.2\left[\frac{18}{100} - \frac{10.67}{100}\right] = 1.35 \times 1.2\,[0.18 - 0.1067] = \mathbf{11.87\ cm.}$$

In other words, as and when this 11.87 cm depth of stored moisture gets consumed by evapotranspiration, irrigation water will be supplied.

From the given consumptive uses, we find that

Water consumed from 1st Nov. to 3 Jan. (when rains start)

= 1.1 mm/day × 30 days (*i.e.*, between 1 Nov. — 30 Nov.)
+ 1.7 mm/day × 31 days (*i.e.*, between 1 Dec.–31 Dec.)
+ 2.4 mm/day × 3 days (*i.e.*, between 1 Jan.–3 Jan.)

$$= 92.9\text{ mm} = \mathbf{9.29\ cm} \qquad ...(i)$$

Hence, water withdrawn from soil during Ist Nov. to 3 Jan. = **9.29** *cm* (< permissible 11.87 cm).

No irrigation is, thus, required till then.

During rains (between 4 Jan. to 19 Jan.), effective rain water received by soil per day

$$= \frac{2.4\text{ cm}}{16\text{ days}} = 0.15\text{ cm/day}$$

Consumptive use of 0.24 cm/day during this period, means that an amount of 0.24–0.15 = 0.09 cm/day of moisture is only consumed from soil, *i.e.*

Additional water consumed from soil during 4 Jan. — 19 Jan.

$$= 0.09\text{ cm/day} \times 16$$

$$= \mathbf{1.44\ cm} \qquad ...(ii)$$

Hence, water withdrawn from soil during 1st Nov.—19 Jan.

$$= (i) + (ii) = 9.29 + 1.44 = 10.73\text{ cm.}$$

Balance water left in soil to be withdrawn before irrigation

$$= 11.87 - 10.73 = 1.14\text{ cm.}$$

This is consumed @ 2.4 mm/day in x days,

$$\text{where } x = \frac{1.14}{0.24} = 4.75\text{ days, say 4 days.}$$

Hence, Ist irrigation will be needed after 4 days from 20th Jan., i.e., on 24th Jan. This irrigation is to be done over 2 days (given), i.e., on 24 Jan. and 25 Jan.

First irrigation water required on 24 Jan. and 25 Jan.

$$= (10.73 + 4 \times 0.24) + \frac{2.4}{10} \text{ cm/day} \times 2 \text{ days}$$

(to compensate for depletion in 2 days)

$$= 11.69 + 0.48 = \mathbf{12.17\ cm.} \quad \textbf{Ans.}$$

With effect from 26 Jan., water is again consumed as below:

Between 26 Jan.–31 Jan.	= 0.24 cm/day × 6 days	= 1.44 cm
Between 1 Feb.–28 Feb.	= 0.15 cm/day × 28 days	= 4.20 cm
Between 1 March–25 March	= 0.35 cm/day × 25 days	= 8.75 cm
	Total	= 14.39 cm
		> 11.87 cm

Hence, another water is required after x days of March, where

$$x = \frac{11.87 - (1.44 + 4.2)}{0.35} = 17.8 \text{ days } \textit{i.e.}, 17 \text{ days}$$

Hence, 2nd irrigation should start on **18th March** and water depth now required is only

$$= 14.39 - 11.87 = \mathbf{2.52\ cm.}$$

Thus, only 2.52 cm irrigation water is required at 2nd time.

Hence, *the required irrigation schedule is*

(*i*) 1st watering on 29th and 30 Jan. = 12.17 cm of water depth
(*ii*) 2nd watering on 18th March = 2.52 cm of water depth **Ans.**

Example 2.19. *A sandy loam soil holds water at 140 mm/m depth between field capacity and permanent witting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 5 mm/day. The area to be irrigated is 60 ha and water can be diverted at 28 l.p.s. The surface irrigation application efficiency is 40%. There are no rainfall and ground water contribution.*

Determine

(*i*) *allowable depletion depth between irrigations.*

(*ii*) *frequency of irrigation*

(*iii*) *net application depth of water*

(*iv*) *volume of water required*

(*v*) *time to irrigate 4 ha plot* (*Engineering Services 1999*)

Solution. Moisture holding capacity of soil = 140 mm/m depth

Depth of root zone = 30 cm = 0.3 m

∴ Moisture holding capacity of root zone

$$= 140 \frac{\text{mm}}{\text{m}} \times 0.3 \text{ m} = 42 \text{ mm} = \mathbf{4.2.\ cm}$$

Allowable depletion = 35%

(*i*) ∴ Available moisture depth or Allowable depletion depth between irrigations

$= 35\% \times 4.2 \text{ cm} = \mathbf{1.47 \text{ cm.}}$ Ans.

Daily use of water = consumptive use = 5 mm/day

(*ii*) ∴ Frequency of irrigation

$$= \frac{\text{Available moisture}}{\text{Moisture consumed per day}} = \frac{1.47 \text{ cm}}{0.5 \text{ cm/day}}$$

$= 2.94$ days., say **3 days. Ans.**

Net water depth to be applied while irrigating each time after 3 days

$= 3 \times 0.5 = \mathbf{1.5 \text{ cm}}$ **Ans.** (in place of 1.47 cm)

Field Irrigation requirement

$$= \frac{\text{Net irrigation requirement}}{\text{Efficiency of irrigation}} = \frac{1.5}{0.4} = 3.75 \text{ cm.}$$

(*iv*) ∴ Qty. of water reqd. in the fields

$= 3.75$ cm of water depth $= 3.75 \text{ cm} \times$ Area of field

$$= 3.75 \text{ cm} \times 60 \text{ ha} = \frac{3.75 \text{ m}}{100} \times (60 \times 10^4) \text{ m}^2 = 22{,}500 \text{ m}^3$$

Hence, vol of water reqd. to irrigate 60 ha area, each time at 3 days interval

$= \mathbf{22{,}500 \text{ m}^3}$ **Ans.**

(*v*) *Time to irrigate 4 ha when irrigation water is supplied @ 28 lps :*

Vol. of water reqd. to irrigate 4 ha plot

$$= 3.75 \text{ cm} \times 4 \text{ ha} = \frac{3.75}{100} \times (4 \times 10^4) \text{ m}^3 = 1500 \text{ m}^3$$

Time during which 1500 m^3 of water can be supplied @ 28 lps.

$$= \frac{1500 \times 10^3 \text{ l}}{28 \text{ lps}} = \frac{1500 \times 10^3}{28} \text{ s}$$

$$= \frac{1500 \times 10^3}{28} \times \frac{1}{60 \times 60} \text{ hr} = \mathbf{14.88 \text{ hr}} \quad \textbf{Ans.}$$

Example 2.20. *Determine the field capacity of a soil for the following data :*

(*i*) *Depth of root zone = 1.8 m*

(*ii*) *Existing moisture = 8%*

(*iii*) *Dry density of soil = 1450 kg/m^3*

(*iv*) *Quantity of water applied to soil = 650 m^3*

(*v*) Water lost due to deep percolation and *evaporation* = 10%

(*vi*) *Area to be irrigated = 1000 m^3* *(AMIE 1999 (Summer) Exam.)*

Solution. Volume of total water applied = 650 m^3.

Water wasted = 10% of 650 m^3 = 65 m^3.

Water used in raising m.c up to field capacity = 650 – 65 = 585 m^3.

Depth of water used in raising m.c up to to field capacity from the existing 8%

$$= \frac{585 \text{ m}^3}{\text{Area} = 1000 \text{ m}^2} = 0.585 \text{ m}$$

But water depth required in root zone of depth *td* increase m.c, is given by eqn.

$$= \frac{\gamma_d}{w}\left[\text{ upper limit mc} - \text{lower limit mc, as fractions }\right]$$

$$\therefore \quad 0.585 = \frac{1.45 \ t/\text{m}^3}{1 \ t/\text{m}^3} \times 1.8 \text{ m}\left[F.C - 0.08\right]$$

or $$(F.C - 0.08) = 0.224$$

or $$F.C = 0.224 - 0.08 = 0.144$$

Hence, Field capacity = **14.4%** **Ans.**

PROBLEMS

1. (*a*) What is meant by 'Duty' and 'Delta' of canal water ? Derive a relationship between duty and delta for a given base period.
   (*b*) Find the delta for sugarcane when its duty is 730 hectares/cumec on the field and the base period of the crop being 110 days. (**Ans.** 130 cm)
   (*c*) Define and explain the following terms as used in relation to water requirements of crops :
   (*i*) Base period. (*ii*) Intensity of irrigation. (*iii*) Cash crops.
2. (*a*) What do you understand by 'Duty' of canal water and what is its importance ? Explain how does duty differs from that at the head of a water-course and that at the head of a canal bringing water to the watercourse.
   (*b*) Mention the approximate values of Duty and Delta for rice, wheat and sugarcane in your region.
3. (*a*) Define 'Duty' and 'Delta', and derive their relationship.
   (*b*) What are the factors on which duty depends ?
   (*c*) How can the duty be improved and what will be the gain ?
   (*d*) What is meant by 'Flow duty' and 'Quantity duty' ?
4. What is meant by 'duty' ?
   Enumerate the different terms by which duty can be improved.
   What are the factors affecting duty ?
   The base period of paddy is 120 days. If the duty for this crop is 900 hectares per cumec, find the value of delta. [**Ans.** 115 cm]
5. Describe briefly the factors affecting duty.
   Water is released at the rate of 5 cumecs at the head sluice. If the duty at the field is 100 hectares/cumec and the loss of water in transit is 30%, find the area of the land that can be irrigated.
   [**Ans.** 350 hectares]
6. What is meant by "Duty of water" ? Explain the influence of several factors which affect duty. What are the different ways in which duty can be expressed ?
   A reservoir with a live storage capacity of 300 million cubic metres is able to irrigate an ayacut of 40,000 hectares with 2 fillings each year. The crop season is 120 days. What is the duty ?
   [**Ans.** 691 hectares/cumec]
7. Name the principal kharif crops of your region, and detail the agricultural and climatic requirements for sowing, growth and harvesting of one of the principal ones. Give the normal requirement of seed per hectare and the average yield per hectare of the crop.
   Suggest ways to increase the "duty" in an irrigation system.

8. (*a*) Explain as how the following factors affect the 'duty' of a crop :

(*i*) Soil and sub-soil condition. (*ii*) Stage of growth.

(*iii*) Temperature. (*iv*) Rainfall.

(*b*) Compute the depth and frequency of irrigation required for a certain crop with data given below :

Root zone depth = 100 cm. Field capacity = 22%

Wilting point = 12% Apparent specific gravity of soil* = 1.50

Consumptive use = 25 mm/day Efficiency of irrigation = 50%

Assume 50% depletion on available moisture before application of irrigation water at field capacity.

[**Hint.** Follow example 2.14, and work out :

Readily available moisture = 5%, and finally work out :

Depth of water stored in root zone = 7.5 cm
Frequency of irrigation = 3 days] **Ans.**

9. Explain with neat sketch the layout of a modern canal system, carrying water from a barrage. Discuss as to how the duty of water increases as we move downstream from the head of the main canal towards the head of the watercourse.

10. Write short notes on :

(*i*) Optimum utilisation of irrigation water.

(*ii*) Crop rotation.

(*iii*) Consumptive use and its estimation.

(*iv*) Water distribution efficiency.

(*v*) Net irrigation requirement (NIR).

(*vi*) Outlet factor.

(*vii*) Estimating depth and frequency of irrigation on the basis of soil moisture regime concept.

(*viii*) Crop seasons in India and their principal crops.

11. Define and explain the following terms :

(*i*) Cash crops. (*ii*) Field capacity.

(*iii*) Available moisture. (*iv*) Soil moisture deficiency.

(*v*) Crop ratio. (*vi*) Overlap allowance.

(*vii*) Paleo irrigation. (*viii*) Kor water depth.

12. How will you proceed for determining the field irrigation requirement (FIR) for an important crop like wheat ? Explain with reference to a sample table, with assumed monthly values of pan evaporations. [**Hint**. Please see Table 2.15]

13. Name any two methods used for estimating consumptive use of water for a particular crop at a particular place. Explain in details the one which is most widely used in your region, and the reasons for preferring that particular method.

---

* **Apparent sp. gr. of soil** $= \dfrac{\gamma_d}{\gamma_w}$, where γ_d is the dry unit wt. of soil (i.e. the soil containing air filled voids).

Actual sp. gr. (S_s or G) $= \dfrac{\gamma_s}{\gamma_w}$, where γ_s is the unit wt. of the soil solids.

3

Canal Irrigation System

3.1. General

A direct irrigation scheme which makes use of a weir or a barrage, as well as a storage irrigation scheme which makes use of a storage dam or a storage reservoir, necessitates the construction of a network of canals, as explained earlier. The entire system of main canals, branch canals, distributaries and minors is to be designed properly for a certain realistic value of peak discharge that must pass through them, so as to provide sufficient irrigation water to the commanded* areas. These canals have to be aligned and excavated either in alluvial soils or non-alluvial soils ; depending upon which they are called *alluvial canals or non-alluvial canals,* as explained below.

3.2. Alluvial and Non-alluvial Canals

(*i*) **Alluvial Soils and Alluvial Canals.** The soil which is formed by transportation and deposition of silt through the agency of water, over a course of time, is called the alluvial soil. Say for example, in the deltaic region** a river carries heavy charge of silt, which gets deposited on the adjoining land, as and when the river overtops its banks during flood season. The process of silt deposition may continue over long periods of time, resulting in the formation of a soil called *Alluvial Soil. The soil which is so formed by the continuous deposition of silt-from the water flowing through a given area, is hence, called the alluvial soil.* The area of alluvial soil is even, and is having a flat surface slope. Hard foundations, are generally not available in this kind of soil. In prehistoric periods, the entire Indo-Gangetic Plain was, perhaps a depression, and was filled up with constant silt deposition dropped from the water flowing through this area, resulting in the formation of an alluvial-soil region. The rivers flowing through such alluvial areas, have a tendency to shift their courses. The river bed consists of sand of considerable thickness, and is, therefore, permeable. Whenever, an irrigation structure is to be constructed on such a river, special precautions and design methods are to be adopted. Most of our North Indian rivers, which pass through alluvial soils, do pose these problems. The canals when excavated through such soils, are called *Alluvial Canals.* Canal irrigation (Direct irrigation using a weir or a barrage) is generally preferred in such areas, as compared to the storage irrigation (*i.e.* by using a dam). Alluvial soil is very fertile, as it can absorb a fair percentage of rainfall and retain it in the substratum, making it highly productive, as water remains available within the root zone of crops.

(*ii*) **Non-alluvial Soils.** Mountaineous regions may go on disintegrating over a period of time, resulting in the formation of a rocky plain area, called non-alluvial area.

* Gross and Net command or commanded area is defined under article 3.6.

** A river before joining the sea gets divided into a number of streams, forming the shape of a delta (Δ), and this region is called the deltaic region.

It has an uneven topography, and hard foundations are generally available. The rivers, passing through such areas, have no tendency to shift their courses, and they do not pose much problems for designing irrigation structures on them. Canals, passing through such areas are called *Non-alluvial Canals*. Major portion of Maharashtra State is non-alluvial. Storage irrigation is preferred to canal irrigation in this type of soil. Non-alluvial soils may be permeable or impermeable, but generally, they are non-permeable.

3.3. Alignment of Canals

Irrigation canals can be aligned in any of the following three ways :

(*i*) as *watershed canal* or *ridge canal.*

(*ii*) as *contour canal* ; and

(*iii*) as *side-slope canal.*

These three types of canals are discussed below :

(*i*) **Watershad Canal or Ridge Canal.** The dividing ridge line between the catchment areas* of two streams (drains) is called the *water-shed*, or the **ridge**. Thus, between two major streams, there is the main watershed (ridge line), which divides the drainage area of the two streams, as shown in Fig. 3.1. Similarly, between a main stream and any of its tributary, there are *subsidiary watersheds* (ridge lines), dividing the drainage between the two streams on either side.

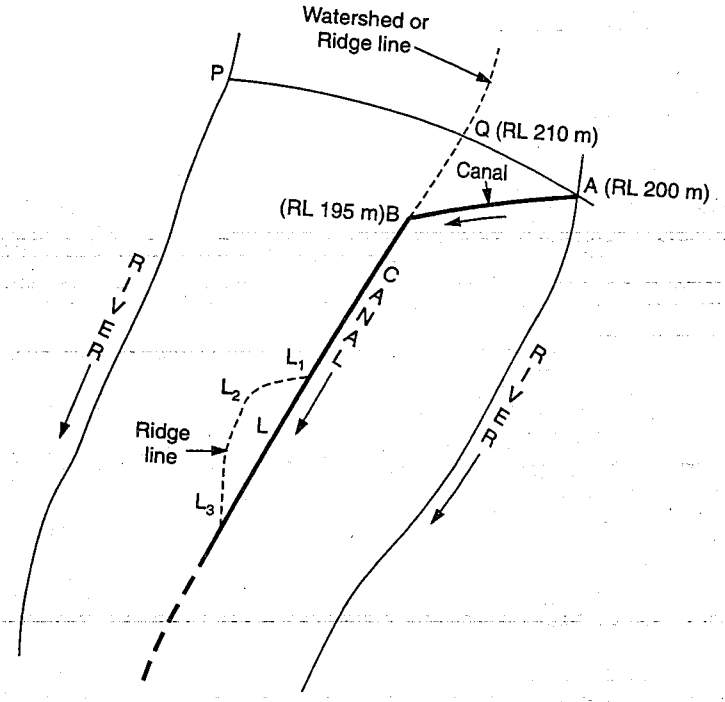


Fig. 3.1. Alignment of a Ridge or Watershed canal
(Head reach of a main canal in plains)

* The area from which rain water flows into a drain or a stream, is known as its *catchment area.*

For a canal system in plain areas, where land slopes are relatively flat and uniform, it is often necessary and advantageous to align canals on the watersheds (ridge lines) of the areas to be irrigated. *The canal which is aligned along any natural watershed (ridge line) is called a watershed canal, or a ridge canal.* The natural limits of the command area of such irrigation channels would be the drainage area on either side of the channel. Aligning a canal (main canal or branch canal or distributary) on the ridge, ensures gravity irrigation on both sides of the canal. Moreover, since the drainage flows away from the ridge, no drainage can cross a canal aligned on the ridge. *Thus, a canal aligned on the watershed saves the cost of construction of cross-drainage works.*

However, the main canal has to be taken off from a river, which is the lowest point in the cross-section, and this canal must mount the watershed (ridge) in as short a distance as possible. Since the available ground slope in the head reaches of a canal is usually much higher than the required canal bed slope, the canal generally needs only a short distance to reach the ridge line. This is illustrated in Fig. 3.1, in which the main canal takes off from a river at point *A*, and mounts the watershed at point *B*. Let the canal bed level at *A* be 200 m and the elevation of the highest point *Q* along the section *PQA* be 210 m. Assuming that the ground slope is 1 m per km, the distance of the point *B* (*RL* 195 m) from *Q* (*RL* 210 m) on the watershed would be 15 km. If the designed canal bed slope is 1 in 4000 (*i.e.* 0.25 m per km), then the length *AB* of the canal would be 20 km. In this length *AB*, the canal would cross small streams, and hence, construction of cross-drainage structures would be necessary for this length. As a matter of fact, the alignment *AB* is influenced considerably by the need of providing suitable locations for the cross-drainage structures. The exact location of *B* would be determined by trial, so that the alignment *AB* results in an economic as well as an efficient canal system. It can also be seen that on the watershed side of the canal *AB*, the ground (*i.e.* area *AQB*) is higher than the ground area on valley side (*i.e.* river side). Therefore, this canal portion *AB* can irrigate only on one side (*i.e.* on left side) of the canal.

When once the canal has reached the watershed (ridge line), it is generally kept on the watershed, except where : (*i*) localities are settled on the watershed ; or (*ii*) where the watershed is looping and not running straight, as shown by $L_1L_2L_3$ in Fig. 3.1. In a situation of a *looping ridge line*, the canal alignment may be taken straight along L_1L_3 by leaving the ridge line. The area between the canal and the watershed in the region *L* (Fig. 3.1) can be irrigated by a distributary which takes off at L_1 and follows the watershed alignment along $L_1L_2L_3$. In the region *L*, the main canal may also have to cross some small streams, and hence some cross-drainage structures may have to be constructed.

If the watershed is passing through villages or towns, the canal may have to leave the watershed (ridge line) for some distance.

The depressions in the ridge line may also necessitate construction of viaducts or syphons to maintain the canal FSL.

(*ii*) **Contour Canals.** The above arrangement of providing the canal along the ridge line are, however, not found economical in hill areas, since the conditions in hills are vastly different compared to those of plains. In hills, the river flows in the valley well below the watershed. Infact, the ridge line (watershed) may be hundred of metres above the river. It therefore becomes virtually impossible to take the canal on top of such a higher ridge line. In such conditions, *contour canals* (Fig. 3.2) are usually constructed.

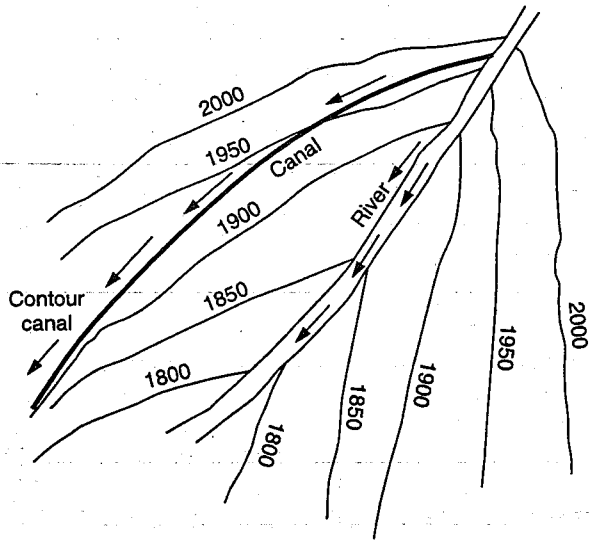


Fig. 3.2. Alignment of a Contour canal.
(Head reach of main canal in hills)

Contour channels follow a contour, except for giving the required longitudinal slope to the canal.

Since the river slope is much steepter than the canal bed slope, the canal encompasses more and more area between itself and the river. It may be noted that more fertile areas in the hills are located at lower levels only.

A contour canal irrigates only on one side because the area on the other side is higher, as can be seen in Fig. 3.2.

As the drainage flow is always at right angles to the ground contours, such a channel would definitely have to cross natural drains and streams, necessitating construction of cross-drainage structures.

(*iii*) **Side slope canal.** A side slope canal is that which is aligned at right angles to the contours ; *i.e.* along the side slopes, as shown in Fig. 3.3. Since such a canal runs parallel to the natural drainage flow, it usually does not intercept drainage channels, thus, avoiding the construction of cross-drainage structures.

In order to finalise the canal networks for an irrigation project, trial alignment of canals are initially marked on the map prepared during the detailed survey. A large-scale map is required to work out the details of individual canals. However, a small-scale map showing the entire command area of the irrigation project is also desirable. The alignents of canals marked on the map are transferred on the field, and adjustments and changes are made, wherever found necessary. These adjustments are transferred on the map as well. The alignment on the field is marked by small masonry pillars erected at every 200 metres distance or so. The centre line on top of these pillars coincides with the exact alighment of

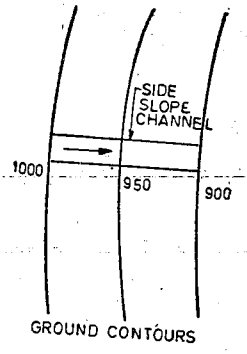


Fig. 3.3. Alignment of a Side Slope canal.

the given canal. In between the adjacent pillars, a small trench may be excavated in the ground, to mark the canal alignment.

3.4. Distribution System for Canal Irrigation

It has been emphasized earlier that the direct irrigation scheme using a weir or a barrage, as well as the storage irrigation scheme using a dam or a reservoir, require a network of irrigation canals of different sizes and capacities. The entire network of irrigation channels (Fig. 3.4) is called the *Canal System.* The canal system, as explained earlier in Chapter 2, consists of :

(*i*) Main canal ;

(*ii*) Branch canals ;

(*iii*) Distributaries, also called *major distributaries* ;

(*iv*) Minors, also called *minor distributaries,*

(*v*) Watercourses.

In case of direct irrigation scheme, a weir or a barrage is constructed across the river, and water is headed up on the upstream side. The arrangement is known as *Head*

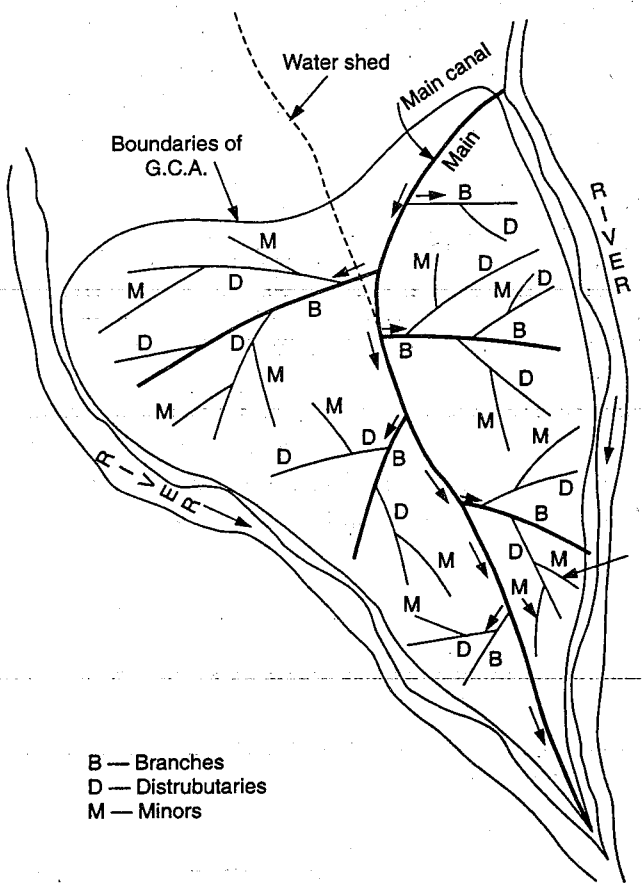


Fig. 3.4. Layout of an Irrigation canal network.

Works or *Diversion Head Works*. It will be explained in details in Chapter 9. Water is diverted into the main canal by means of a diversion weir. A head regulator is provided at the head of the main canal, so as to regulate the flow of water into the main canal.

In storage irrigation scheme, a dam is constructed across the river, thus forming a reservoir on the upstream side of the river. The water from this reservoir is taken into the main canal through the outlet sluices. There are generally two main canals, which off-take from the reservoir from the left side or the right side, and are hence called the *Left Bank Canal* and the *Right Bank Canal*, respectively, as shown in Fig. 3.5(*a*).

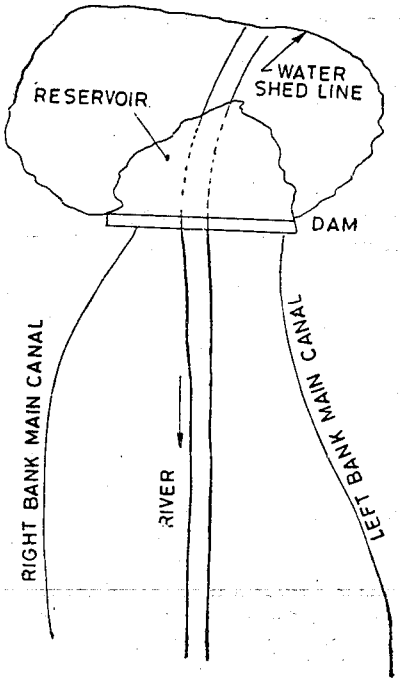


Fig. 3.5(*a*). Storage Irrigation Scheme.

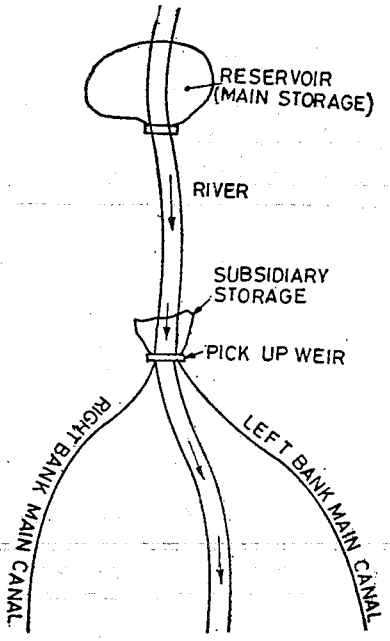


Fig. 3.5(*b*). Storage Irrigation Scheme with Pick up weir.

It is, however, not at all necessary that the main canal(s) take off from the reservoir, *and infact, they rarely do so*. No canal has taken off from Bhakra reservoir. In most storage schemes, canals, usually take off from a *pick up weir or a barrage*, located downstream of the reservoir, as shown in Fig. 3.5(*b*). The main canal in such a case will take off from the upstream side of the pick up weir, just as in a normal diversion weir scheme.

The requirement of construction of such a *pick up head works* may become necessary when the irrigation command area is far away from the dam site, and constructing the main canal(s) from the reservoir upto the beginning of the command area may be a waste and a useless exercise. Secondly, in some cases, the headwork might have been constructed first as a part of a direct (non-storage) irrigation scheme, and the upstream reservoir may have to be constructed on a later date, in accordance with its own priority or necessity, when the natural lean season flows become insufficient to meet the irrigation and other water demands of the downstream area. Such a reservoir would add some

new irrigation command and firm up the irrigation in the existing command. That is what happened in the case of Bhakra dam, where canal system was already in existence as a part of a non storage scheme, purely based on perennial river supplies. Similar constructions are expected in case of Yamuna river, where *Western Yamuna Canal (WYC)* and *Eastern Yamuna Canal (EYC)* taking off from the **Tajewala headworks*** have been in existence for the last more than 100 years, but without the backing of any upstream reservoir. Such a direct irrigation scheme proved satisfactory in olden times, when population was less and the river used to get sufficient snowmelt throughout the lean season. However, with the increasing population and consequently increasing water demand along with dwindling snowmelt due to extension of habitat in upper hills and green house effects of urbanisation, the available natural flows in lean season have become insufficient, necessitating construction of reservoir(s) upstream. That is why, there are plans to construct three reservoirs at *Renuka, Kishau* and *Lakhwar-Vyasi* in the Yamuna catchment upstream. When these dame are constructed, there will be very little addition to irrigation command area, but they will mostly firm up the irrigation in the already existing *WYC* and *EYC* command, and make available water for other needs. The (Hathnikund) headworks constructed as a part of direct irrigation scheme will then, become a part of a storage irrigation scheme.

It can be easily understood that a reservoir redistribute the water in time (storing water in rainy season and releasing it in lean season) ; while the barrage and the canal system will redistribute it in space, taking it upto the fields.

In both these irrigation schemes, when once the water reaches into the main canal, the problem left is to distribute this water upto the fields. The purpose is achieved through a network of channels, as described below :

(*a*) **Main Canal (Head reach).** The canal headworks are generally situated on the river flowing in a valley, and the canal should reach the ridge line in the shortest possible distance. The canal, in this reach, must, therefore, be aligned very carefully, and has to be generally excavated in deep cuttings below N.S.L. (natural surface level). Sometimes, it has to cross various drainage lines. Many a times, straight alignment has to be sacrificed and detours need to be accepted, in order to achieve a good site for cross drainage works.

(*b*) **Main Canal (Portion below head reach.)** Attempts are made to align the canal along the ridge and somewhat central to the command area. Sometimes, ridge line has to be sacrificed, to bypass towns and villages, etc. Main canal is not required to do any irrigation.

(*ii*) **Branch Canals.** *Branch canals* are taken off from the main canal on eather side to take irrigation water to the whole tract required to be irrigated. Very little irrigation is infact, done from the branch canals themselves, as they serve to supply water primarily the distributaries. Attempts are made to align them along subsidiary ridges. Discharge in a branch channel, is generally, more than 30 cumec.

(*iii*) **Distributaries.** Smaller channels which take off from the branch canals and distribute their supply through outlets into minors or water courses, are called distributaries. They are aligned either as ridge canals or as contour canals. Discharge in a distributary is generally less than 30 cumec.

(*iv*) **Minors.** Sometimes, the country is such that the distance between the distributary outlet and the farmer's field is very long ; say more than 3 km or so. In such

* The age old Tajewala headworks has recently been replaced by newly constructed Hathnikund berrage headworks.

a case, small channels called minors, are taken off from the distributaries, so as to supply water to the cultivators at the point nearer to their fields. Discharge in a minor, is generally, less than 2.5 cumec.

(*v*) **Watercourses.** These are not the government channels and belong to the cultivators. They are small channels, which are excavated and maintained by the cultivators at their own costs, to take water from the government-owned outlet points, provided in the distributary or the minor.

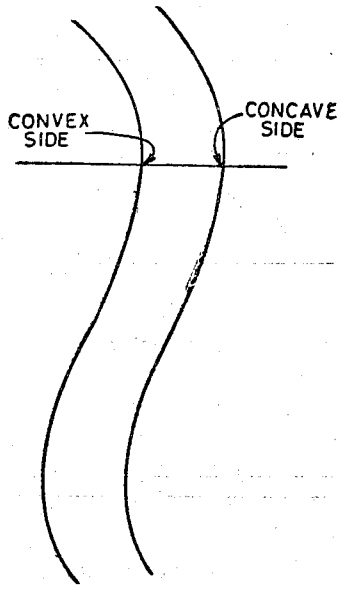


Fig. 3.6.

3.5. Curves in Channels

Attempts are made to align the channels straight as far as possible. But many a times, the curves become inescapable. Whenever, a curve is proposed, while aligning unlined channels, it should be as gentle as possible. A curve causes disturbance of flow and results in silting on the inside (*i.e.* convex side) and scouring on the outside (*i.e.* concave side).

Pitching is, therefore, sometimes proposed on the concave side, so as to avoid scouring. If the discharge is more, the curve should be more gentle and should, therefore, have more radius. The generally adopted minimum values of radii for different discharges are tabulated below in Table 3.1.

Table 3.1

Discharging Capacity of Channels (in cumecs)	*Minimum recommended curve Radius (in metres)*
Over 100	1500
30—100	900
15—30	600
3—15	300
0.5—3	150
Less than 0.5	100

3.6. Certain Important Definitions

Before we discuss the techniques, which are employed to determine the 'design discharge' for a canal, let us first of all, define certain important terms, which often come across in the design of irrigation canal systems.

3.6.1. Gross Command Area (G.C.A.). It is the total area, bounded within the irrigation boundary of a project, which can be economically irrigated without considering the limitation of the quantity of available water. *It includes the cultivable as well as the un-cultivable area. For example, ponds, residential areas, roads, reserved forests, etc. are the uncultivable areas of the gross command area.*

A given irrigation canal system lies in a *doab* (*i.e. the area between two drainages*), and can economically irrigate the *doab*. It is, obviously, uneconomical to use the irrigation system to irrigate across the two drainages. Thus, the boundaries of gross command of a canal system is, fixed by the drainages on both sides of the main irrigation canal, as shown in Fig. 3.4. The gross command area, evidently represents the geographical area of the *doab*.

3.6.2. Culturable or Cultivable Command Area (CCA). Culturable area is the cultivable part of the gross command area, and includes all land of GCA on which cultivation is possible. *It will, thus, include pastures and fallow lands, which can be made cultivable.* Obviously, it does not include uncultivable part of the gross command, like populated areas, ponds, roads, reserved forests, ushar land, etc.

At any given time, however, all the cultivable land may not be actually under cultivation. Therefore, sometimes, the CCA is divided into two categories ; *i.e.* (*i*) *cultivated portion of CCA* ; and (*ii*) *Cultivable but not cultivated portion of CCA*. In the absence of detailed data, CCA may be assumed to be equal to 80% of GCA.

3.6.3. Intensity of Irrigation (Seasonal and Annual). The entire cultivated portion of the culturable command area (CCA) is not proposed to be irrigated at one time (in one season) to avoid **intensive irrigation** of a particular area, which may cause harmful effects like water logging*, salinity and malaria, etc. Moreover, due to shortage of irrigation water, larger area of the command is usually covered with partial coverage of the fields of different sub-areas or pockets. *Some land of a particular sub area is thus, either allowed to take rest, or is shown with crops which do not require irrigation water.* Fields left unirrigated in one season will be supplied water in the next season, when some other fields will be left unirrigated. The fields are thus, supplied water in rotation over different crop seasons, thereby irrigating only about 40 to 60% of the fields of various sub areas over a season. The irrigation water is, thus, usually supplied extensively, covering a larger area. Such as **extensive irrigation**, covering a larger portion of the cultivable command, is preferred to the *intensive irrigation* of the smaller portion of the command, to avoid general famine conditions over some area (which would be totally left unirrigated, if the entire water is used in intensive irrigation of some other portion). Extensive irrigation also helps in avoiding harmful effects of over-irrigation (like salinity and water-logging), which is caused by intensive irrigation.

Due to these reasons, only a small percentage of CCA is brought under irrigation over a given season. *This percentage of CCA proposed to be irrigated in a given season is called the intensity of irrigation of that season, or seasonal intensity of irrigation.* Say for example, the sanctioned intensity of irrigation under Bhakra canal system is only 27.6% for Kharif season, and 34.4% for Rabi season.

Sometimes, the intensity of irrigation is worked out over the entire year (inclusive of two or more crop seasons), when it is called the *annual irrigation intensity* or *annual intensity of irrigation* (AII), which may be defined as the percentage of CCA which may be irrigated annually. AII is, thus, obtained by dividing the *gross irrigated area* (*i.e.* total area irrigated once in the year + the area irrigated more than once in that year) by the CCA. The annual intensity of irrigation is the sum total of intensities of irrigation of all the seasons of the year. The annual intensity of irrigation for Bhakra canal system

* Explained in details in chapter 6.

is thus, computed by adding the intensity of irrigation for kharif season and that for the Rabi season, giving combined annual irrigation intensity of 27.6 + 34.4 = 62%.

The annual irrigation intensity is usually found to be in the range of 40 to 60%, but needs to be raised to the range of 100 to 180% by cultivating larger parts of CCA with more than one crop in a year, and through improved management and economical utilisation of the available irrigation water. From the perusal of col. (7) of table IV under the first chapter on "Introduction to the subject", it can be seen that in India, U.P. and Punjab, States have already exceeded this figure of AII beyond 100%, to 139.6% and 133.4%, respectively, with Haryana (90.2%) and West Bengal (73.3%) following these two best irrigated States. Other States are far behind to such an extent that the overall annual intensity of irrigation in the country is only 45.9%.

3.6.4. Net and Gross Sown Areas. Sometimes, two crops in two seasons are grown during a particular year on the same area. Hence, such an area will be sown more than once during a given year. If this area is added to the area which is sown only once (and called the *net shown area*), then we get what is known as the *gross sown area*, or the *gross cropped area*. Hence,

Gross cropped or Gross sown area (during a year)

= Net cropped area, *i.e.* area sown once in a year
+ Area shown more than once during the same year
...(3.1)

3.6.5. Net and Gross Irrigated Areas. Based on the above analogy, the area which is irrigated once during a year is called the *net irrigated area*, and when to this is added the area irrigated more than once, we obtain the *gross irrigated area.*

∴ Gross irrigated area (in a given year)

= Net irrigated area (*i.e.* area irrigated once in a year)
+ Area irrigated more than once during the same year.
...(3.2)

3.6.6. Area to be Irrigated. The area proposed to be irrigated in any one crop season or over any given year, is called the *area to be irrigated in that season or the year*, respectively. It is obtained by multiplying CCA by the seasonal or annual intensity of irrigation, as the case may be. The areas to be irrigated are usually worked out separately for each crop season, because the water requirement of the crops of two seasons are quite different.

3.6.7. Time Factor. To check the dangers of over irrigation, leading to water-logging and salinity, no distributary is allowed to operate on all the days during any crop season. *The ratio of the actual operating period of a distributary to the crop period is called the time factor of the distributary.* For the Bhakra canal system, for example, the time factors for Kharif and Rabi seasons are fixed at 0.80 and 0.72, respectively, which means that each distributary would receive its full supply for a period of $0.80 \times 180 = 144$ days, and $0.72 \times 180 = 129$ days, respectively, in each crop season of 180 days.

For computing the design capacity of a distributary, therefore, the computed water requirement (for the crops proposed to be grown and irrigated by that distributary) should be divided by the time factor, since this factor is less than 1.

3.6.8. Capacity Factor. The capacity factor for a canal is the ratio of the mean supply discharge in a canal during a period to its designed full capacity. Since canals have to run almost to their full designed capacities during Kharif (summer) season, this value usually varies from 0.9 to 0.95 for Kharif season. However, since water requirement during Rabi (winter) season reduces to about $\frac{2}{3}$rd times* the full supply, the capacity factor usually varies from 0.60 to 0.70 for Rabi season. To improve this factor during Rabi season, the cropped area in Rabi season is, hence, usually increased.

3.6.9. Full supply coefficient. Full supply coefficient is the design duty at the head of the canal. In other words, the number of hectares irrigable per cumec of the canal capacity at its head, is known as the full supply coefficient of the canal. It can hence, be represented by the equation :

Full supply coefficient

$$= \frac{\text{Area estimated to be irrigated during base period}}{\text{Design full supply discharge at the head of the canal}} \qquad ...(3.3)$$

This factor is also called the **Duty on capacity.**

3.6.10. Nominal Duty. It is the ratio of the area actually irrigated by the cultivators to the mean supply discharge let out from the outlet of the distributary over the crop period.

For example, let x cumec of water is released daily from the outlet of a distributary for 100 days (says) in a total crop period of 125 days (say). Then, the mean supply discharge over the crop period will be $\frac{x \times 100}{125} = 0.8x$ (cumec).

If the area of crop irrigated by this discharge is A hectares, then the nominal duty will be given as :

$$\text{Nominal duty} = \frac{A(ha)}{0.8x \text{ (cumec)}} = 1.25 \frac{A}{x} \text{ ha/cumec.}$$

3.7. Computing the Design Capacity of an Irrigation Canal

Whenever one plans for supplying irrigation water, one has to think of the likely crops that would be sown in any one season. The peak rate of water requirement of all the crops in each season of a year is also needed to be worked out. The capacity of the canal should be sufficient to fulfil the maximum of the peak demand of all the crops that are required to be irrigated at any one time amongst all the seasons. It is explained below in details:

The most important Rabi crop is *wheat,* which requires water from *December to March,* during the Rabi season. Similarly, *Paddy* (Rice) is the most important Kharif crop, requiring water from *June to November.* So it can be presumed that when Rabi crops require water, Kharif crops do not, and *vice-versa.* Sugarcane and garden crops are **perennial crops**, requiring water throughout the year. Hence, the canal may be designed for a capacity equal to the greater of the water requirement of Rabi and Sugar

* The winter (Rabi) crops usually mature on about 52% of the supply required by crops in summer ; but due to lower winter discharges in canals, the percentage of losses enroute becomes higher in winter season. The actual water supply at the head of canal is, therefore, found to be about 66% of full supply.

cane (plus garden crops if any) ; or Kharif and Sugarcane (and garden crops, if any). This is a very simple method for fixing the capacity of the channel. The entire Rabi area is supposed to be sown with wheat and entire Kharif area with Paddy ; Sugarcane area (including garden crop area, if any) is excluded from both. Their water requirements in cumec are worked out for both seasons, separately. The water requirement of sugarcane and garden crops are separately worked out each, in cumec. The sum of cumec required by the Rabi plus Sugarcane (including garden crop, if any) and Kharif plus Sugarcane (including garden crop if any), are separately worked out. The canal capacity may then be fixed for the maximum of the two values.

The most important point which must be kept in mind while fixing the channel capacity is that, we must take into account the **keenest demand** of the crop and not the **average demand.** For example, let Rice require 120 cm of water during 120 days, thus giving an average outlet factor of 864 hectares/cumec

$$\left(i.e.\ D = \frac{864\,B}{\Delta} = \frac{864 \times 120}{120} = 864 \right)$$

But a canal designed on this average outlet factor will prove to be very inadequate, as it will fail to supply the required water to the crop at its **peak demand**, *i.e.* at the time of **kor-watering**, as explained below :

The **kor depth** for Rice is about 19 cm and the **kor period** is about 2 weeks (*i.e.* 14 days). It means that 19 cm of water depth must be supplied in about 14 days. The outlet factor for this, works out to be 637 hectares/cumec as

$$D = \frac{864\,B}{\Delta} = \frac{864 \times 14}{19} = 637$$

Now, discharge required to mature A hectares of land for an outlet factor of 864 is $\frac{A}{864}$ cumecs ; while that for an outlet factor of 637 is $\frac{A}{637}$ cumecs. Out of these two values, the second value $\left(i.e.\ \frac{A}{637} \right)$ is more, and hence, the discharge required for fulfilling the kor demand of the crop is more than that for the average demand.

Similarly, wheat requires about 40 cm of water in a total base period of about 160 days; thus giving an average outlet factor of 3464 hectares/cumec. But the kor water depth required by wheat is about 14 cm in about four weeks, giving an outlet factor of 1728 hectares/cumec. Applying the previous reasoning, we can say that the discharge required to fulfil kor-demand is much more than that for the average-demand (almost double of average). Hence, it follows that the peak demand, *i.e.* kor-demand of the crop should be taken into account while fixing the capacity of a canal. Moreover, the provision for canal losses should also be made, while deciding the final capacity of the canal.

The method, described above, for determining the canal capacity, is a simplified approximate process. To be more precise, we can find out the monthly or fortnightly water requirements of various crops (as was explained in Chapter 2). The water depth required in this interval is multiplied by the crop area, so as to give the volume of water required in this interval. Dividing the volume by interval, we can find out the discharge required in each interval, by various crops. The summation of which for all the crops will give us the discharge required by all the crops in each interval. The canal may then be designed for maximum of these values.

It is evident that in order to be more precise, the interval should be as small as possible. Generally, monthly water requirement studies are conducted, and the canal capacity is increased by 20 to 25% to cater for the peak demand in the month.

Example 3.1. *The gross commanded area for a distributary is 6000 hectares, 80% of which is culturable irrigable. The intensity of irrigation for Rabi season is 50% and that for Kharif season is 25%. If the average duty at the head of the distributary is 2000 hectares/cumec for Rabi season and 900 hectares/cumec for Kharif season, find out the discharge required at the head of the distributary from average demand considerations.*

Solution. G.C.A. = 6000 hectares

$$\text{C.C.A.} = 6000 \times \frac{80}{100} = 4800 \text{ hectares.}$$

Area to be irrigated in Rabi season

$$= \text{C.C.A.} \times \text{Intensity of Irrigation}$$

$$= 4800 \times \frac{50}{100} = 2400 \text{ hectares.}$$

Area to be irrigated in Kharif season

$$= 4800 \times \frac{25}{100} = 1200 \text{ hectares.}$$

Water required at the head of the distributary to irrigate **Rabi** area

$$= \frac{2400}{2000} \text{ cumecs} = \mathbf{1.20\ cumec.} \qquad ...(i)$$

Water required at the head of the distributary to irrigate **Kharif** area

$$= \frac{1200}{900} \text{ cumecs} = \mathbf{1.33\ cumec.} \qquad ...(ii)$$

Thus, the requirement in Kharif season is 1.33 cumec and that in Rabi season is 1.20 cumecs. The required discharge is maximum of the two, *i.e.* **1.33 cumec. Ans.**

Hence, the distributary should be designed for 1.33 cumec discharge at its head, from average demand considerations. The head regulator should be sufficient to carry 1.33 cumec ; and in Rabi season, only 1.20 cumec will be released.

Example 3.2. *Determine the discharge required at the head of the distributary in Example 3.1 given above, for fulfilling maximum crop requirement. Assume suitable values of kor depth and kor period.*

Solution. Let us assume a kor period of 4 weeks for Rabi (wheat) and 2.5 weeks for Kharif crop (rice). Also assume, Kor depth of 13.5 cm for Rabi (wheat) and 19 cm for Kharif (rice) crop.

$$\text{Now, outlet factor for rabi} = \frac{864 \times B}{\Delta} = \frac{864 \times (4 \times 7)}{13.5} = 1792 \text{ hectares/cumec.}$$

$$\text{Outlet factor for Kharif} = \frac{864 \times (2.5 \times 7)}{19} = 796 \text{ hectares/cumec.}$$

Area to be irrigated in Rabi season (worked out in previous example)

$$= 2400 \text{ hectares}$$

Area to be irrigated in Kharif season (worked out in previous example)

$$= 1200 \text{ hectares.}$$

Water reqd. at the head of the distributary to irrigate Rabi area

$$= \frac{2400}{1792} = 1.34 \text{ cumec} \qquad ...(i)$$

Water required at the head of the distributary to irrigate Kharif area

$$= \frac{1200}{796} = 1.51 \text{ cumec.} \qquad ...(ii)$$

The required discharge is maximum of the two, *i.e.* **1.51 cumec. Ans.**

Note : The required discharge from kor demand considerations have gone up to 1.51 cumec from 1.33 cumec (worked out in the previous Example), *i.e.* an increase of about 14%.

Example 3.3. *The culturable commanded area of a watercourse is 1200 hectares. Intensities of sugarcane and wheat crops are 20% and 40% respectively. The duties for the crops at the head of the watercourse are 730 hectares/cumec and 1800 hectares/cumec, respectively. Find (a) the discharge required at the head of the watercourse (b) determine the design discharge at the outlet, assuming a time factor equal to 0.8.*

Solution. C.C.A. = 1200 hectares

Intensity of irrigation for sugarcane = 20%

$$\therefore \text{ Area to be irrigated under sugarcane} = 1200 \times \frac{20}{100} = 240 \text{ hectares}$$

Intensity of irrigation for wheat = 40%

$$\therefore \text{ Area to be irrigated under wheat} = 1200 \times \frac{40}{100} = 480 \text{ hectares}$$

Duty for sugarcane = 730 hectares/cumec

Duty for wheat = 1800 hectares/cumec.

$$\therefore \quad \text{Discharge required for sugarcane} = \frac{240}{730} \text{ cumec} = 0.329 \text{ cumec}$$

and $$\text{Discharge required for wheat} = \frac{480}{1800} \text{ cumec} = 0.271 \text{ cumec.}$$

Now, sugarcane requires water for all the 12 months and wheat requires water for only rabi season. Hence, the water requirement at the head of the watercourse at any time of the year will be the summation of the two, *i.e.* equal to $0.329 + 0.271 = 0.6$ cumec.

(*a*) Hence, the discharge required at the head of the water-course is **0.6 cumec. Ans.**

Note : The discharge during rabi season will be 0.6 cumec and for the rest of the year, it will be 0.329 cumec.

(*b*) Time factor = 0.8 ; since the channel runs for fewer days than the crop days, therefore, the actual design discharge at the outlet

$$= \frac{0.6}{0.8} = 0.75 \text{ cumec.} \quad \textbf{Ans.}$$

Example 3.4. *The culturable commanded area for a distributary is 15,000 hectares. The intensity of irrigation (I.I.) for Rabi (wheat) is 40% and for Kharif (rice) is 15%. If the total water requirement of the two crops are 37.5 cm and 120 cm and their periods of growth are 160 days and 140 days respectively ; (a) Determine the outlet discharge from average demand considerations ; (b) Also determine the peak demand discharge, assuming that the kor water depth for two crops are 13.5 cm. and 19 cm. and their kor periods are 4 weeks and 2 weeks respectively.*

Solution. C.C.A. = 15,000 hectares.

I.I. For wheat (Rabi) = 40%

I.I. For rice (Kharif) = 15%

Wheat (Rabi) area = $15{,}000 \times 0.40 = 6000$ hectares

Rice (Kharif) area = $15{,}000 \times 0.15 = 2250$ hectares

Δ for wheat = 37.5 cm.

Δ for rice = 120 cm.

B for wheat = 160 days

B for rice = 140 days

Now $$D = \frac{864\,B}{\Delta}$$

Average duty (D) for wheat $= \dfrac{864 \times 160}{37.5} = 3686$ hectares/cumec

Average duty (D) for rice $= \dfrac{864 \times 140}{120} = 1008$ hectares/cumec

Outlet discharge required for wheat $= \dfrac{\text{Area}}{\text{Duty}} = \dfrac{6000}{3686} = 1.63$ cumec

Outlet discharge required for rice $= \dfrac{2250}{1008} = 2.23$ cumec.

The required design discharge at outlet (from average demand considerations) is maximum of the two values, *i.e.* **2.23 cumec. Ans.**

(*b*) Kor water depth for wheat = 13.5 cm.

Kor period for wheat = 4 weeks = $4 \times 7 = 28$ days

Kor water depth for rice = 19 cm.

Kor period for rice = 2 weeks = $2 \times 7 = 14$ days.

Duty for wheat (for kor demand) $= \dfrac{864 \times B}{\Delta}$

$$= \frac{864 \times \text{Kor period for wheat}}{\text{Kor water depth for wheat}} = \frac{864 \times 28}{13.5} = 1792 \text{ ha/cumec}$$

Duty for rice (for Kor demand)

$$= \frac{864 \times \text{Kor period for rice}}{\text{Kor water depth for rice}} = \frac{864 \times 14}{19} = 636 \text{ ha./cumec}$$

Outlet discharge required for wheat (for kor demand)

$$= \frac{\text{Area}}{\text{Duty}} = \frac{6000}{1792} = 3.35 \text{ cumec.} \qquad ...(i)$$

Outlet discharge required for rice (for kor demand)

$$= \frac{2250}{636} = 3.54 \text{ cumec} \qquad ...(ii)$$

The required design discharge at the outlet, from peak demand considerations, is maximum of these two values, *i.e.* = **3.54 cumec. Ans.**

Example 3.5. *At a certain place in India, the transplantation of rice takes 16 days, and the total depth of water required by the crop is 60 cm on the field. During this transplantation period of 16 days, rain starts falling and about 10 cm of rain is being utilised to fulfil the rice demand. Find the duty of irrigation water required for rice during transplanting period. (a) Assuming 25% losses of water in water courses, find the duty of water at the head of the watercourse. (b) Find the duty of water at the head of distributary, assuming 15% losses from the distributary head to the watercourse head.*

Solution. Total depth of water required for transplanting rice = 60 cm.

Useful rainfall = 10 cm.

Extra water depth required to be supplemented by irrigation = 60 – 10 = 50 cm.

Period in which this water is required = 16 days.

$$\text{Duty of irrigation water} = \frac{864\,B}{D} = \frac{864 \times 16}{50} = \mathbf{276.5 \text{ hectares/cumec}}$$

(*a*) Assuming 25% losses in the watercourse, we have

Duty at the head of watercourse

$$= 277.5 \times 0.75 = 207.4 \text{ hectares/cumec.} \quad \textbf{Ans.}$$

(*b*) Similarly, duty at the head of the distributary (assuming 15% losses)

$$= 207.4 \times 0.85 = 176.3 \text{ hectares/cumec.} \quad \textbf{Ans.}$$

Example 3.6. *A main canal, which offtakes from a storage reservoir, has to irrigate crops in a certain country having three seasons in a year. Data for the irrigated crops, is given in table 3.2:*

Table 3.2

S. No.	*Name of crop*	*Crop period in days*	*Area to be irrigated in hectares*	*Duty at the head of the main canal in hectares/cumec*
1. (*a*)	Sugarcane	280	315	630
(*b*)	Overlap for Sugarcane in hot weather	100	70	630
2.	Jowar (Rabi)	120	4800	1600
3.	Bajri (Monsoon)	120	5600	2800
4.	Vegetables (Hot season)	120	350	700

(*a*) *Find the discharge required at the head of the main canal, taking time factor for the main canal as 0.7.*

(*b*) *What should be the gross storage capacity of the reservoir ? Assume suitable factors, wherever needed.*

Solution. Water required by various crops, is as follows (figures taken from Table 3.2).

(*i*) Water required for sugarcane (whole year) $= \frac{315}{630} = 0.5$ cumec.

(*ii*) Water required for overlap sugarcane (Hot season) $= \frac{70}{630} = 0.11$ cumec.

(*iii*) Water required for jowar (Rabi season) $= \frac{4800}{1600} = 3.0$ cumecs

(*iv*) Water required for bajri (Monsoon season) $= \frac{5600}{2800} = 2.0$ cumecs.

(*v*) Water required for vegetables (Hot season) $= \frac{350}{700} = 0.5$ cumec.

Total water required in each season is as follows :

(*a*) in hot season $= 0.5 + 0.11 + 0.5 = 1.11$ cumec ;

(*b*) in monsoon season $= 0.5 + 2.0 = 2.5$ cumec ;

(*c*) in Rabi season $= 0.5 + 3.0 = 3.5$ cumec.

It is evident that the maximum water is required in Rabi season, *i.e.* equal so **3.5 cumec.**

Now, Time factor = 0.7 ;

Therefore, full supply discharge (based on average demand or duty) $= \frac{3.5}{0.7} = 5.0$ cumec.

Assuming the peak demand discharge to be 25% more than the average, the full supply discharge on peak demand 5×1.25 cumec = 6.25 cumec.

(*a*) Hence the F.S.Q. at the head of the main canal (assuming negligible seepage losses from the head of the main canal to the fields)

= 6.25 cumec **Ans.**

(*b*) To find out the gross storage capacity of the reservoir, let us work out the volume of water required by various crops as follows :

1. Water required by sugarcane $= 0.5 \times 280 \times 24 \times 60 \times 60$ m^3

(*i.e.* discharge × days × secs in one day)

Water required by overlap sugarcane $= 0.11 \times 100 \times 24 \times 60 \times 60$ m^3

Water required by jowar $= 3.0 \times 120 \times 24 \times 60 \times 60$ m^3

Water required by bajri $= 2.0 \times 120 \times 24 \times 60 \times 60$ m^3

Water required by vegetables $= 0.5 \times 120 \times 24 \times 60 \times 60$ m^3

Total water required

$$= \Sigma = 24 \times 60 \times 60\,[0.5 \times 280 + 0.11 \times 100 + 3.0 \times 120 + 2.0 \times 120 + 0.5 \times 120]$$

$$= 24 \times 60 \times 60\,[140 + 11 + 660] \text{ cubic metres}$$

$$= (24 \times 60 \times 60 \times 811) \text{ cubic metres} = 70.07 \times 10^6 \text{ cubic metres}$$

$$= 70.07 \text{ million cubic metres} = \mathbf{70.07\ M.m^3}$$

Assume reservoir losses due to absorption and evaporation as 10% of 70.07, *i.e.* = 7.01 M.m^3.

To allow for late and irregular monsoon, let us assume a carry over storage, equal to 5% of 70.07 M.m^3, *i.e.* 3.35 M.m^3.

$\therefore$ The live storage of reservoir = 70.07 + 7.01 + 3.35 = **80.43 mcm**

Now, Gross storage = Live storage + Dead storage.

Assuming dead storage at 10% of gross storage, we get gross storage

$$= G = 80.43 + 0.1 \times \text{Gross storage}$$

or $$G = 80.43 + 0.1\,G \quad \therefore \quad 0.9\,G = 80.43$$

or $$G = \frac{80.43}{0.9} = 89.4 \text{ M.m}^3.$$

Hence, the Gross storage of the reservoir is **89.4 million cubic metres.** **Ans.**

Example 3.7. *A pump is installed on a well to lift water and to irrigate rice crop, sown over 3 hectares of land. If duty for rice is 864 hectares/cumec on the field and pump efficiency is 48% ; determine the minimum required input (H.P.) of the pump, if the lowest well water level is 8 metres below the highest portion of the field. Assume negligible field channel losses.*

Solution. Area of rice to be irrigated = 3 hectares.

Duty of water for rice = 864 hectares/cumec.

Discharge required for rice for fulfilling its duty demand

$$= \frac{3}{864} \text{ cumec.} = \frac{1}{288} \text{ cumec.}$$

$\therefore$ Volume of water lifted per second $= \frac{1}{288}$ m^3

$\therefore$ Weight of water lifted per second $= \frac{1}{288}$ m$^3 \times 9.81$ kN/m^3 $= 0.0341$ kN/sec.

($\because$ unit wt. of water = 9.81 kN/m^3)

Minimum static lift of pump = 8 metres

$\therefore$ Work done by the pump in lifting this water

$$= 0.0341 \text{ kN/sec.} \times 8 \text{ m} = 0.273 \text{ kN.m/sec.} = 0.273 \text{ kWatt.}$$

The output of the pump (H.P.) $= \frac{0.273}{0.735} = 0.37$ ($\because$ 1 metric H.P. = 0.735 k.watt)

$\therefore$ Input H.P. of the pump $= \frac{\text{Output}}{\eta} = \frac{0.37}{0.48} = 0.77$; **say 0.8 H.P. Ans.**

Example 3.8. *Monthly water requirement studies as shown in Table 3.3 were conducted on various crops that are required to be grown.*

Table 3.3

Month	Field irrigation requirement in cm (FIR)					
	Paddy	Groundnut	Maize	Cotton	Sugarcane	Chillies
(1)	(2)	(3)	(4)	(5)	(6)	(7)
June 1—30	19.3				25.9	
July 1—31	6.0				7.6	
Aug. 1—31	7.4				6.8	
Sept. 1—30	7.9				6.0	1.7
Oct. 1—31	29.9		3.4	4.0	34.7	23.5
Nov. 1—30	20.7	6.3	15.1	8.0	42.3	22.4
Dec. 1—31		16.2	23.8	20.6	18.0	16.6
Jan. 1—31		21.6	20.5	22.8	22.0	10.8
Feb. 1—28		13.4		14.3	25.0	
March 1—31					36.5	
April 1—30					40.8	
May 1—31					50.0	

A reservoir is proposed to be constructed to command an area equal to 1,20,000 hectares. The various crops are : Paddy, Groundnut, Maize, Cotton, Sugarcane and Chillies. The areas under irrigation of these crops are going to be : 20%, 5%, 5%, 10%, 10%, 3% of command respectively. Determine the annual storage required for the reservoir, assuming canal losses as 25% of head discharge, and reservoir evaporation and dead storage losses as 20% of gross capacity.

Solution. The areas to be irrigated for Paddy, groundnut, Maize, Cotton, Sugarcane and Chillies respectively are : 24,000, 6,000, 6,000, 12,000, 12000, and 3,600 hectares. The water required at field *i.e.* at outlet point is worked out for each crop (monthwise) as shown in Table 3.4. The table is self explanatory.

Table 3.4

Month	Water required in hectare-metre (i.e. Area× Depth in metres given in question)					
	Paddy	Groundnut	Maize	Cotton	Sugarcane	Chillies
	24000 hectares	6000 hectares	6000 hectares	12000 hectares	12000 hectares	3600 hectares
(1)	(2)	(3)	(4)	(5)	(6)	(7)
June 1—30	4632				3081,	
July 1—31	1440				912,	
Aug. 1—31	1776				816,	
Sept. 1—30	1896				720	61.2
Oct. 1—31	7176		204	480	4164	846.0
Nov. 1—30	4968	378	906	960	5076	806.4
Dec. 1—31		972	1428	2472	2160	597.6
Jan. 1—31		1296	1230	2736	2640	388.8
Feb. 1—28		804		1716	3000,	
Mar. 1—31					4380,	
April 1—30					4896,	
May 1—31					6000,	
Σ	21888	3450	3768	8364	37845	2700

Grand total = 78,015 hectare metres

Water required at outlet = 78,015 ha.m

Water required at canal head $= \frac{78015}{0.75} = 1,04,020$ ha.m

Gross storage required $= \frac{1,04,020}{0.80} = 1,30,025$ ha.m

$= 1,30,025 \times 10^4 \text{ m}^3 \approx \mathbf{1300\ M.m^3}$

Example 3.9. *Fix the channel capacity for the crop pattern of example 3.8. Assume any suitable data you need.*

Solution. Table 3.4 is extended in Table 3.5, so as to show the monthly water requirement of all the crops in col. (8). Discharge required in each interval is woked out in col. (9). The table is otherwise self-explanatory.

Table 3.5

Month	*Paddy*	*Groundnut*	*Maize*	*Cotton*	*Sugarcane*	*Chillies*	*Total vol. of water reqd. by all crops (V) in ha m = summation of col. (2) to (7)*	*Discharge reqd. in ha. m/day = col. 8 / interval in days*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
June 1—30	4632				3081		7713	257.1
July 1—31	1440				912		2352	75.8
August 1—31	1776				816		2592	83.5
September 1—30	1896				720	61.2	2677.2	89.2
October 1—31	7176		204	480	4164	846.0	12870	415
November 1—30	4968	378	906	960	5076	806.4	13094.4	**436.48***
December 1—31		972	1428	2472	2160	597.6	7629.6	246
January 1—31		1296	1230	2736	2640	388.8	8290.8	267
February 1—28		804		1716	3000		5520	197
March 1—31					4380		4380	142
April 1—30					4896		4896	163
May 1—31					6000		6000	193

*Maximum value.

The maximum value, out of all the discharges in col. (9) of table 3.5 is 436.48 ha-m/day

$$= \frac{436.48 \times 10^4}{24 \times 60 \times 60} \text{ cubic metre per second}$$

$$= \frac{436.48 \times 10^4}{86400} = \frac{436.48}{8.64} = \mathbf{50.6\ cumec.}$$

Let us assume that the peak demand of the month is 1.25 times the average demand.

Then, the peak demand of the most demanding month

$= 1.25 \times 50.6 = 63.25$ cumecs, say 64 cumec

Hence, the channel capacity at the outlet point.

= 64 cumec. Ans.

Note. The above channel capacity is at the outlet point, as the FIRs were taken into consideration.

Example 3.10. *A reservoir is proposed to be constructed to command an area of 1,20,000 hectares. It is anticipated that ultimately sugar and paddy would both be irrigated equal to 20% of the command and Rabi equal to 50% of the command, making a total annual irrigation equal to 90% of the commanded area.*

(a) Work out the storage required for the reservoir, assuming water requirement as prevailing in any area you are familiar with. Assume canal losses at 25% of the head discharge, and reservoir evaporation and dead storage losses at 20% of gross capacity.

(b) For the above crop pattern, fix the channel capacity in the head reaches.

Solution. Total commanded area = 1,20,000 hectares

Area under Sugar = $0.20 \times 1,20,000 = 24,000$ hectares

Area under Paddy = $0.20 \times 1,20,000 = 24,000$ hectares

Area under Rabi = $0.50 \times 1,20,000 = 60,000$ hectares

Let us assume water depth requirements, average duties, on capacity duties, etc. for the three crops, as shown in Table 3.6.

Table 3.6

Crop	*Total water depth reqd. in cm*	*Av. Duty in hectares/cumec*	*Duty on capacity, i.e. max demand duty in hectares/cumec*
(1)	*(2)*	*(3)*	*(4)*
Sugar	90	2600	600
Paddy	120	864	637
Rabi	40	3464	1728

The quantity of water required for all crops is worked out as shown in Table 3.7.

Table 3.7

Crop	*Δ in cm (assumed)*	*Hectares under cultivation*	*Water reqd. in hectare-metres = col. (2)× col. (3)/100*
(1)	*(2)*	*(3)*	*(4)*
Sugar	90	24000	21600
Paddy	120	24000	28800
Rabi	40	60000	24000
			Σ = 74400

Total volume of water required at outlet

$= 74400$ hectare metres $= 74400 \times 10^4$ cubic metres

$= 744$ million cubic metres (M.m^3)

Canal losses = 25% of head discharge (given)

∴ Total volume of water required at the head of the canal system

$$= \frac{744}{0.75} \text{ M.m}^3 = 992 \text{ M.m}^3$$

or Net Reservoir Storage = **992 M.m^3**

Reservoir evaporation and dead storage loss = 20% of Gross storage *(given)*

(*a*) Gross Storage of the required reservoir $= \frac{992}{0.80} = \mathbf{1240\ M.m^3}$ **Ans.**

(*b*) *Channel capacity.* Channel capacity can be fixed on the basis of 'on capacity outlet factors' assumed in col. (4) Table 3.6. Let us work out the capacity discharge required for each crop as shown in col (4) of Table 3.8.

Table 3.8

Crop	*'Capacity Duty' hectares/cumec (assumed)*	*Hectares under cultivation (given)*	*Capacity in cumecs* $= \frac{col.\ (3)}{col.\ (2)}$
(1)	*(2)*	*(3)*	*(4)*
Sugar	600	24000	40
Paddy	637	24000	37.7
Rabi	1728	60000	34.7

Cumecs reqd. for Sugar and Paddy = 40 + 37.7 = 77.7 cumec

Cumecs reqd. for Sugar and Rabi = 40 + 34.7 = 74.7 cumec.

Assuming that the Paddy does not require water when Rabi requires, and Sugar may require its peak demand at a time when either Rabi or Paddy is in full demand ; we can fix the capacity of the channel as maximum of these two, *i.e.* **77.7 cumec.** (It is very hypothetical). In practice, however, demand of sugar may arise only when Rabi or Paddy are in very small demand. In such a hypothetical case, assuming that the peak demands of Sugar and Rabi or Paddy (one of them) may arise at the same time, we may fix channel capacity as :

Channel capacity at the outlet = 77.7 cumec

$\therefore$ Channel capacity in the head reaches $= \frac{77.7}{0.75}$ cumec $= \mathbf{103.6\ cumec.}$ **Ans.**

3.8. Losses of water in canals

During the passage of water from the main canal to the outlet at the head of the watercourse, water may be lost either by evaporation from the surface or by seepage through the peripheries of the channels. These losses are sometimes very high, of the order of 25 to 50% of the water diverted into the main canal. In determining the designed channel capacity, a provision for these water losses must be made. The provision for the water lost in the watercourses and in the fields is however, already made in the outlet discharge factor, and hence, no extra provision is made on that account. Evaporation and seepage losses of channels are discussed below :

(1) **Evaporation.** The water lost by evaporation is generally very small, as compared to the water lost by seepage in certain channels. Evaporation losses are generally of the order of 2 to 3 per cent of the total losses. They depend upon all those factors*, on which the evaporation depends, such as temperature, wind velocity, humidity, etc. In summer season, these losses may be more but seldom exceed about 7% of the total water diverted into the main canal.

* Discussed in article 7.34.2.3.

(2) **Seepage.** There may be two different conditions of seepage, *i.e.* (*i*) Percolation, (*ii*) Absorption.

(*i*) *Percolation.* In percolation, there exists a zone of continuous saturation from the canal to the water-table and a direct flow is established. Almost all the water lost from the canal, joins the ground water reservoir.

The loss of water depends upon the difference of top water surface level of the channel and the level of the water-table. (*i.e.* *H*) as shown in Fig. 3.7.

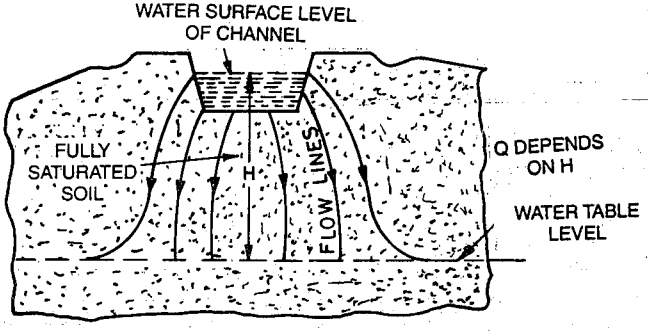


Fig. 3.7. Percolation.

(*ii*) *Absorption.* In absorption, a small saturated soil zone exists round the canal section, and is surrounded by zone of decreasing saturation. A certain zone just above the water-table is saturated by capillarity. Thus, there exists an unsaturated soil zone between the two saturated zones, as shown in Fig. 3.8.

In this case, the rate of loss is independent of seepage head (*H*) but depends only upon the water head *h* (*i.e.* distance between water surface level of canal and the bottom of the saturated zone) plus the capillary head h_c, as shown in Fig. 3.8.

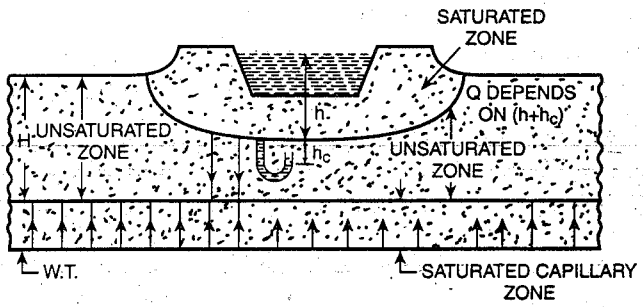


Fig. 3.8. Absorption.

The seepage losses depend upon the following factors :

(*i*) Type of seepage, *i.e.* whether 'percolation' or 'absorption'.

(*ii*) Soil permeability.

(*iii*) The condition of the canal ; the seepage through a silted canal is less than that from a new canal.

(*iv*) Amount of silt carried by the canal ; the more the silt, lesser are the losses.

(*v*) Velocity of canal water ; the more the velocity, the lesser will be the losses.

(*vi*) Cross-section of the canal and its wetted perimeter.

For designs, a combined figure for seepage losses as well as for evaporation losses, expressed as cumec per million sq. m of wetted perimeter, may be taken, as tabulated in Table 3.9.

Table 3.9. Channel Losses

S. No.	*Type of soil through which channel is excavated*	*Total loss in cumec/million sq. m of wetted area*
1.	Rocks	0.9
2.	Black cotton soil	1.6
3.	Alluvial red soil	2.5
4.	Decayed rock, gravel, etc.	3.0
5.	Loose sandy soil	5.5

3.8.1. Empirical formulas for channel losses. The channel losses can be determined by using certain empirical formulas, such as

(*a*) $$\Delta Q = \frac{1}{200}(B + D)^{2/3} \quad ...(3.4)$$

where ΔQ = Channel losses in cumecs per km length of channel,

B = Bed width of the channel in metres.

D = Depth of water in the channel in metres.

This formula is generally used in U.P.

(*b*) $$\Delta Q = 1.9 \cdot Q^{1/6} \quad ...(3.5)$$

where ΔQ = Losses in cumecs per million sq. m. of wetted perimeter.

Q = Discharge in cumecs.

The above formula is commonly used in Punjab.

3.9. Canal Regulation

The amount of water which can be diverted from the river into the main canal depends upon :

(*i*) The water available in the river ;

(*ii*) The capacity of the main canal ; and

(*iii*) the share of other canals taking off from the river.

The flow from the main canal is further diverted to the branch canals ; and from the branch canals to the distributaries. The distribution of water (flow) depends upon the water demand of various canals. *The method of distribution of available supplies into various canals of a canal system is called the canal regulation.*

Obviously, when there exists a significant demand of water anywhere in the command area of a canal, the said canal has to be kept flowing. The canal can, however, be closed, if the water demand falls below a specified quantity. The canal shall be reopened again when the water demand exceeds the specified minimum quantity. Normally, there always exists a demand in some part of the command area of any major canal. Such major canals can, hence, be closed only for a very small period (say hardly for three to four weeks in a year). These canals, thus run almost continuously. They may, however, carry discharges much less than their full capacity, either when there is less demand, or when the available supplies are insufficient.

When the demand in less, only the distributaries which need water are kept running, while others (including those which have very little demand) are closed. In case of higher demand but insufficient supplies, either all the smaller channels are made to run simultaneously and continuously with reduced supplies ; or some channels are closed turn by turn and the remaining ones are made to run to their full or near-full capacities. The first alternative of simultaneously running all the distributaries with reduced supplies is generally not preferred, as it causes channel siltation, weed growth, increased seepage, water-logging and low heads on the outlets. The second alternative of running the distributaries turn by turn with full supplies is, hence, generally preferred, since it does not have these disadvantages, and allows sufficient time for inspections and repairs of the channels.

A roster is, therefore, usually prepared for indicating the allotted supplies to different channels, showing the schedule of their closure and operation.

A *flexible regulation* is always preferred, so that the supplies are allocated in accordance with the anticipated demand. The allocation of supplies should hence be varied on the basis of the information provided by the canal revenue staff, who is supposed to keep a close watch on the crop condition and irrigation water demand of their respective distributaries and outlets.

The discharge in a canal is usually regulated at its *head regulator**, which is generally designed as a meter, as to indicate the discharge being let out from it. However, when a head regulator cannot be used as a discharge meter, a depth guage is provided at about 200 m distance downstream of the head regulator. The gauge-discharge curve of the canal is kept ready, as to indicate the discharge for the observed gauge reading. By manipulating the opening through the regulator gates, the desired discharge can be obtained in the given canal.

3.10. Distribution of Water into the Fields Through Water Courses

The water from a distributary or a minor is allowed to flow into the water course, through an opening, called *module* or *outlet**. When once the water reaches the water course, the problem of its equitable distribution among the various farmers or fields arises. The release of water into the water course and its sharing by farmers with different field holdings, largely depends upon the available supply. Since the water supply is usually limited, the following two possible alternatives for its distribution become available :

(1) The canal irrigation is restricted to a limited area, which can be fully supported with the lowest available supply. This does not lead to optimum utilisation of available water and may cause *intensive irrigation* and its ill effects.

Agricultural production and protection against famine would also not be optimum. The production may be maximum per unit of land covered (if the farmers donot over-irrigate), though it would certainly not be per unit of water available. This approach would, however, not require a precise or sophisticated method for distribution of irrigation water amongst the field owners. The delivery system for this alternative can be either continuous or demand-based, depending upon the availability of water. A continuous delivery system can be effectively used for large farms and continuous terraced

* Discussed in details in chapter 13.

rice fields. Though ideal, a demand-based delivery system is not practical on large irrigation systems.

(2) In the second and usually adopted alternative, irrigation is extended to a much larger area than what could be supported by the lowest available supply. This *extensive irrigation often creates perpetual scarcity of irrigation water but ensures that a comparatively much less quantity of water remains unutilised. Agricultural production and protection again famine will be at optimum levels. The crop production would be maximum per unit of available water, though it may not be optimum per unit of land covered.* Since this method of delivering water is in the interest of a larger section of farmers, it is usually adopted in our country, inspite of the fact that this method of distribution requires elaborate control and monitoring on the release of water from the different outlets into the different water courses, and to further ensure that the farmers in possession of different land holdings (land areas) do share water as per their decided shares and utilise it on the decided days, rather than permitting the upstream farmers to utilise larger amounts of irrigation water and thereby leaving very little or no water for farmers at the tail end of the conveyance system. Such a system of distributing irrigation water among the farmers is popularly known as the Warawandi, and is explained below :

3.10.1. Warabandi. *Warabandi* (Fig. 3.9) is an integral management system for conveying and distributing irrigation water from the source (river or reservoir) down to the farm gates (*i.e. nakkas*) of various land holders, so as to ensure supplies upto the tail end of the command area. The entire water conveyance system is divided into the following three categories :

(*i*) *Primary distribution system* ;

(*ii*) *Secondary distribution system* ; *and*

(*iii*) *Tertiary distribution system.*

Conveyance of water from the source into the *main canal*, feeding two or more *branch canals* (which may operate by rotation) and may not carry the total required supply, constitute what is known as the **primary distribution system**. This primary distribution system runs throughout the irrigation season with varying supply.

The **secondary distribution system** consists of a large number of distributaries which run by rotation but usually carry full supply. They are fed by the branch canals of the primary distribution system. The water is finally released from the distributary into the water courses through the outlets (modules). The secondary distribution system ends at this outlet point.

The water flowing into the *water course or it branches*, from an outlet of a distributary, is finally allocated to various fields situated along the water course by a *time roster*. This is the **tertiary distribution system.**

Whereas, the primary and secondary distribution systems are fully managed and controlled by the State Irrigation Department, the tertiary distribution system (downstream of the outlet) is managed by the farmers, in a classic example of the *joint state-farmer management* of the irrigation system.

In this warabandi system, each unit of culturable cammand area (CCA) is allocated a certain rate of flow, called **water allowance**, *whose value is generally a comprise between the likely demand and the supply for a given project. The carrying capacities of the watercourses and the distributaries are designed on the basis of this decided value*

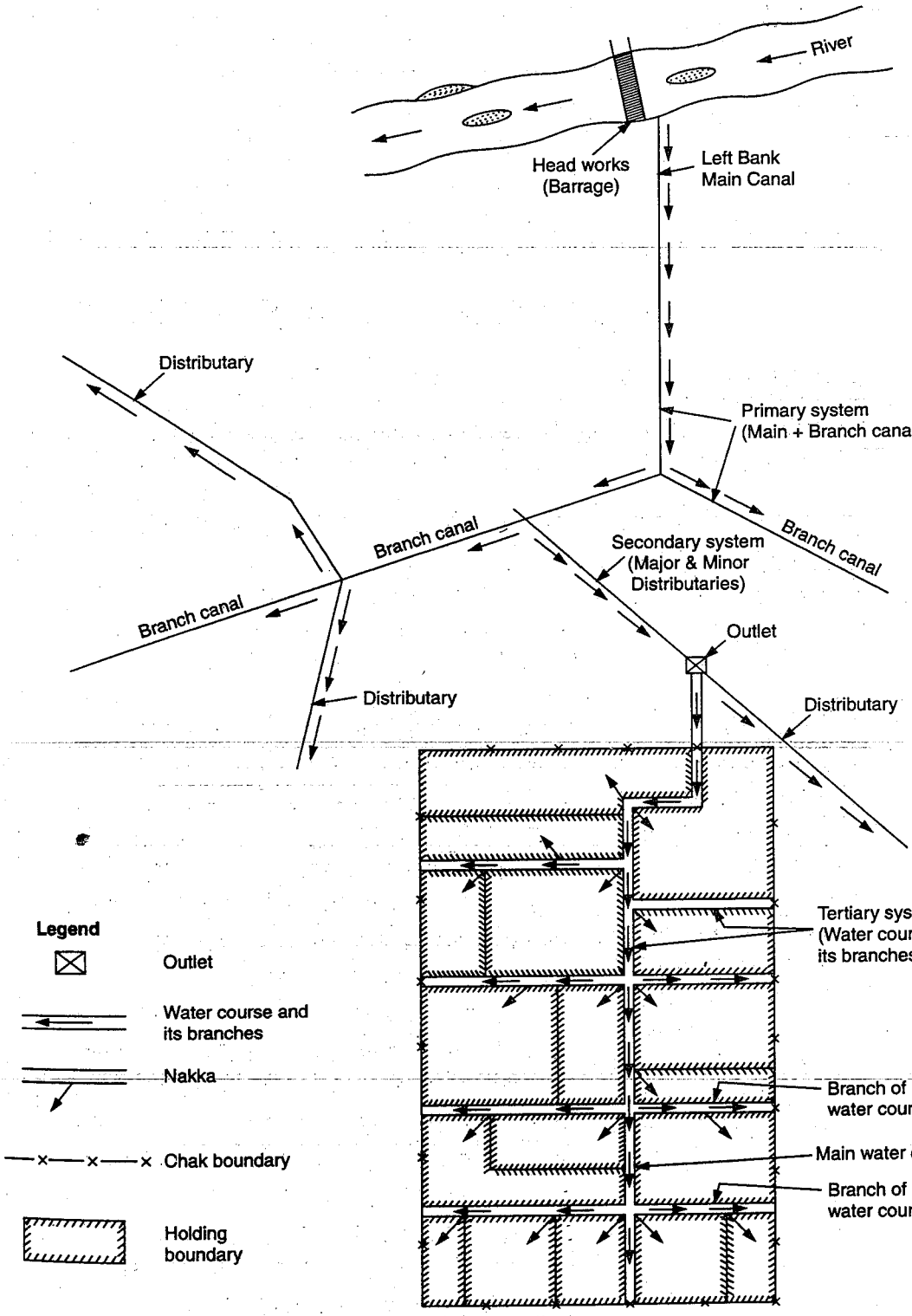


Fig. 3.9. Typical warabandi distribution system.

of water allowance. Say for example, under the Bhakra canal system, covering Punjab, Haryana and Rajasthan, the value of *water allowance* at the head of water courses is fixed @ 0.017 cumec per 100 hectares of culturable command area.

Whenever distributaries run, they are expected to carry their full supply. The outlets to water courses are so planned and constructed that all the water courses on a distributary withdraw their authorised shares of water simultaneously. The design capacities of distributaries and water courses in this warabandi system are, thus, based on the hectares of culturable command area (which is fixed), rather than on the basis of water demand of variable cropping pattern adopted by the different farmers. The warabandi system, thus, does not provide greater amount of water to the farmers, who are growing more water consuming crops in their fields. The system, therefore, does not provide any undue benefits to some particular farmers, as it imposes equal water scarcity on every user. Such distribution of water, in proportion to the land holdings of different farmers, by relating the design capacities of watercourses and distributaries to the CCA rather than to the cropping pattern, is obviously advantageous, as it offers equitable distribution of water, avoiding unnecessary rivalary and ill will among the farmers.

Under the warabandi system, the distributaries are operated at full capacities*, usually on **eight-day periods**. The number of these periods would depend on the availability of water and crop requirements. In a normal year, it is possible to run the distributaries of the Bhakra project for 18 periods during Kharif season and 16 periods during Rabi season.

The distribution of water from the water course, being managed by the farmers, is done on **seven-day rotation basis**, with the help of an *agreed roster* (called **roster of turns**), which divides $24 \times 7 = 168$ hours of seven days in the ratio of the land holdings of various farmers. The eight day running period of distributary ensures a minimum of seven days running for each water course, including those which are at the tail end of the distributary.

Each cultivator's right to share water in a water course is guaranted by law, as the canal Act empowers the canal officers (AE's, EE's and SE's) to ensure this right for everyone.

Whenever a distributary is running, a water course receives its share of water at a constant rate round the clock and water distribution proceeds from head to tail. Each cultivator on the watercourse is entitled to receiving the entire water in a water course only on specific week day and at a specific time (including night time). There is no provision in the system to compensate a defaulting farmer, who has failed to utilise his turn for any reason.

Roster of Turn. The cycle of turns on a *water course or its branch* starts from its head, proceeds downstream and ends at the tail. Before a farmer receives his share of water, some time is spent in filling the water-course length between the point of his taking over and the beginning of his holding. This time is called *bharai* (*i.e.* the filling time), which is debited to a common pool time of 168 hours and credited to the account of the concerned farmer. The supply in the water course has to be stopped when the tail end farmer is having his turn, since the water filled in the water-course during the entire *bharai* period can be discharged only into the field of the tail-end farmer ; and hence,

* Running the distributaries and water courses at full capacities, reduces their running time, thereby reducing conveyance losses, besides checking siltation, weed growth, etc.

normally, the total time spent on filling the entire length of the water course should be recovered from him in lieu of this ; but then, he does not receive this water (flowing after closure of outlet) at a constant rate. Since such a supply, beyond a limit, is not efficient for field applications, the tail end farmer is compensated for it, and is allowed a certain discount on the recovery of the *bharai* time. This value of *bharai* is called *Jharai* (*i.e.* emptying time). Evidently, precise determination of *Jharai* time is not feasible, and its present values are favourable to the tail-end farmers. After allowing for *bharai* and *jharai* timings, the flow time (*FT*) for a unit area of land holding of an individual farmer is given as :

Flow time (*FT*) for unit area in hours (*h*)

$$= \frac{168h - [\text{Total bharai time in h} - \text{Total jharai time in h}]}{\text{Total area to be irrigated by the given outlet or watercourse}} \quad ...(3.6)$$

Flow time in hour for an individual farmer

= [*FT* for unit area × area of farmer's fields + (his *bharai* – his *jharai*)] ...(3.7)

Obviously, *bharai* is usually zero in the case of last farmer (at tail) and *jharai* is zero for all except the last farmer. It may also be noted that the losses in the watercourse are not reflected anywhere in the above equations.

PROBLEMS

1. (*a*) Differentiate between "Alluvial" and "Non-alluvial" canals.
   (*b*) What are the different ways in which the irrigation canals can be aligned ?
   (*c*) Name the various types of canals which are required to be constructed while planning a canal irrigation system.
2. Give an account of the investigations and surveys required while planning an irrigation canal project in a given tract of land. Discuss the factors governing the selection of alignment of the main canal and its branches.
3. (*a*) "The construction and design of canals in Tamil Nadu is easier compared to that in U.P." Discuss critically the above statement with reference to the type of soils available in these two States of India.
   (*b*) State how will you fix up the following in an irrigation canal :
   (*i*) Alignment ; (*ii*) Full supply level ; (*iii*) Full supply discharge.
4. (*a*) Discuss the factors that are considered in fixing the alignment of a main canal in a Deltaic area.
   (*b*) The left bank canal of a storage irrigation scheme carries a discharge of 10 cumecs and has a culturable commanded area of 8000 hectares. The intensity of Rabi crop is 70% and base period is 110 days. The right bank canal of the scheme on the other hand, carries a discharge of 24 cumecs and commands a culturable area of 15000 hectares. The intensity of rabi crops is 80% and the base period is 110 days. Compare the efficiencies of the two canal systems.
5. (*a*) Define and explain the following terms :
   (*i*) Culturable commanded area. (*ii*) Intensity of irrigation
   (*iii*) Nominal duty (*iv*) Full supply coefficient
   (*v*) Time factor (*vi*) Capacity factor.
   (*b*) The discharge available from a tubewell is 136 cubic metres per hour. Assuming 3000 hours of working for this tubewell in a year, estimate the culturable area that this tubewell can command. The intensity of irrigation is 50% and the average water depth required for the rabi and kharif crops is 51 cm

[**Hint.** $\text{CCA} = \frac{136 \times 3000}{0.51} \cdot \frac{1}{0.5} \cdot \frac{1}{10^4}$ hectares = 160 hectares. **Ans.**]

6. Write short notes on :

(*i*) Alluvial and Non-alluvial soils.

(*ii*) Water-shed canals and contour canals

(*iii*) Distribution system for canal irrigation

(*iv*) Water losses in irrigation canals.

(*v*) Curves in irrigation channels.

7. (*a*) Explain in details, as to how will you proceed for determining the approximate value of design discharge of an irrigation canal. What procedure are required for its precise computations ?

(*b*) Discuss the precautions which you would adopt while designing irrigation channels aligned on curvilinear paths.

8. (*a*) How is the capacity of an irrigation channel decided for Indian irrigation conditions? Also discuss, as to why the kor demand ofcourse should be considered in place of average demand while computing this capacity.

(*b*) A field channel has a culturable commanded area of 2000 hectares. The intensity of irrigation for gram is 30% and for wheat is 50%. Gram has a crop period of 18 days and kor depth of 12 cm, while wheat has a kor period of 15 days and a kor depth of 15 cm. Calculate the discharge of the field channel.

[**Hint.** Duty for gram $= \dfrac{864 \times 18}{12} = 1296$ ha/cumec

Duty for wheat $= \dfrac{864 \times 15}{15} = 864$ ha/cumec

Area of gram irrigated $= 2000 \times 0.30 = 600$ ha.

Discharge for gram $= \dfrac{600}{1296}$ cumec $= 0.46$ cumec

Area of wheat irrigated $= 50\% \times 2000 = 1000$ ha

Discharge for wheat $= \dfrac{1000}{864} = 1.16$ cumecs

Total discharge required $= 0.46 + 1.16 = 1.62$ cumecs. **Ans.**]

9. A watercourse commands an irrigation area of 800 hectares. The intensity of irrigation of rice in this area is 50%. The transplantation of rice crop takes 15 days and total depth of water required by the crop is 60 cm on the field during the transplantation period, given that the rain falling on the field during this period is 15 cm. (*i*) Find the duty of irrigation water for the crop on the field during transplantation ; (*ii*) at the head of the distributary, assuming losses of water to be 20% in the water courses (*iii*) Calculate the discharge required in the watercourse.

[**Ans.** (*i*) 216 hectare/cumec (*ii*) 172.8 hectare/cumec, (*iii*) 2.32 cumecs.]

10. How is the water lost while flowing through irrigation canals ? What are the factors affecting these losses, and how will you estimate these losses ? How do these losses affect the duty of water and what precautions will you undertake in order to minimise them ?

11. What are the possible causes of water losses in a canal ? What are the methods adopted for reducing such losses ?

4

Sediment Transport and Design of Irrigation Channels

Whenever water flows in a channel (natural or artificial), it tries to scour its surface. Silt or gravel or even larger boulders are detached from the bed or sides of the channel. These detached particles are swept downstream by the moving water. This phenomenon is known as *Sediment Transport.*

4.1. Importance of Sediment Transport

(*i*) The phenomenon of sediment transport causes large scale scouring and siltation of irrigation canals, thereby increasing their maintenance. Many poorly designed artificial channels get silted up so badly, that they soon become inoperable, causing huge economic loss to the public exchequer. The artificial channels should, therefore, be properly designed, and should not fail to carry the sediment load admitted at the canal headworks.

(*ii*) The design and execution of a flood control scheme is chiefly governed by the peak flood levels, which, in turn, depend upon the scour and deposition of sediment. Firstly, the bed levels may change by direct scouring or deposition of sediment, and thereby changing the flood levels. Secondly, the scouring and silting on the river banks may create sharp and irregular curves, which increase the flow resistance of the channel, and thereby, raising the flood levels for the same discharge.

(*iii*) Silting of reservoirs* and rivers is another important aspect of sediment transport. The storage capacity of the reservoir is reduced by its silting, thereby, reducing its use and life. Natural rivers used for navigation are frequently silted up, reducing the clear depth (draft) required for navigation. Sediment deposited in these rivers and harbours may often require costly dredgings.

Sediment transport, thus, poses numerous problems, and is a subject of great importance, and possesses enough potential for further research and development.

4.2. Sediment Load

The sediment in a canal is a burden to be borne by the flowing water, and is, therefore, designated as sediment load.

Bed Load and Suspended Load. The sediment may move in water either as bed load or as suspended load. Bed load is that in which the sediment moves along the bed with occasional jumps into the channel. While, the suspended load is the one in which the material is maintained in suspension due to the turbulence of the flowing water.

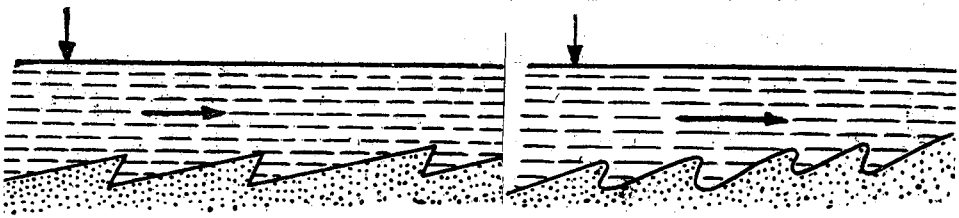
* For details of reservoir silting, please see Chapter 13.

4.3. Bed Formation (Practical Aspect)

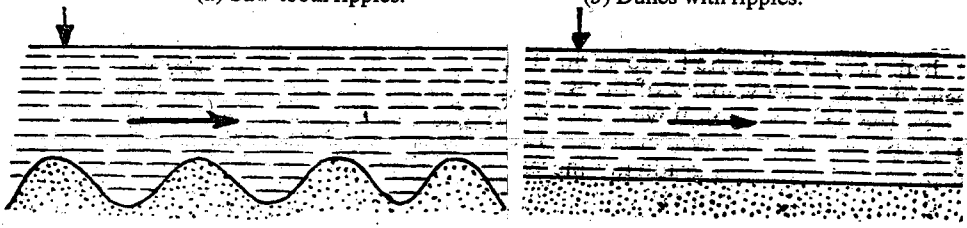
The channel bed may be distorted into various shapes by the moving water, depending upon the discharge or the 'velocity of the water.

At low velocities, the bed does not move at all, but it goes on assuming different shapes as the velocity increases. *Let us see what happens to a channal bed made of fine sand (lesser than 2mm dia) when the velocity is gradually increased in steps.*

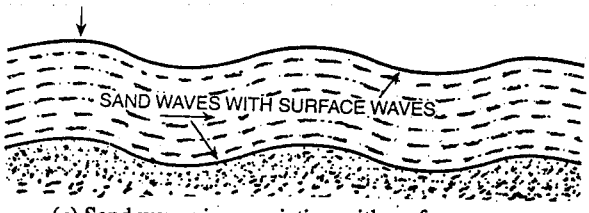
When the velocity is gradually increased, then first of all, a stage is reached, when the sediment load comes just at the point of motion. This stage is known as **threshold stage of motion.** On further increase of velocity, the bed develops *ripples* of the saw-tooth type, as shown in Fig. 4.1 (*a*). Such ripples can also be seen in sand on any beach. As the velocity is increased further, larger periodic irregularities appear, and are called *Dunes.* When they first appear, ripples are superimposed on them [Fig. 4.1 (*b*)]. But at still higher velocities, the ripples disappear and only the dunes are left [Fig. 4.1 (*c*)]. Dunes may form in any grain size of sediment, but ripples do not occur if the size



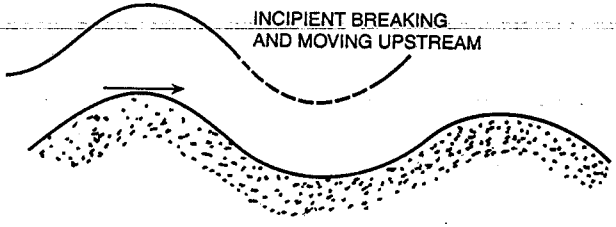
(*a*) Saw-tooth ripples. (*b*) Dunes with ripples.



(*c*) Dunes. (*d*) Flat surface.



(*e*) Sand waves in association with surface waves.



(*f*) Antidunes.

Fig. 4.1. Different shapes of bed developed along river flow with gradual increase in discharge or flow velocity.

of the bed particles is coarser than 0.6 mm. Dunes are much larger (in length and height) and more rounded than ripples. So much so that ripples seldom exceed 40 cm in length (between two adjacent troughs) and 4 cm in height (trough to crest), while the dunes in laboratory flumes may have 3m length and height up to about 40 cm, while in large rivers, they may be as high as 15 metres, with several hundred metres length. Crests of both, do not extend across the entire width of the stream, *i.e.*, both formations tend to occur in the form of "short crested waves". The flow conditions remain sub-critical in both these regimes. While most of the sediment particles move along the bed, some finer particles of sediment may go in suspension.

When the velocity is increased beyond formation of dunes, the dunes are erased by the flow, leaving very small undulations or virtually a flat surface with sediment particles in motion [Fig. 4.1 (*d*)]. Further increase in velocity, results in the formation of sand waves in association with surface waves [Fig. 4.1 (*e*)]. As the velocity is further increased, so as to make the Froude number $\left(i.e.\ \frac{V}{\sqrt{gy}}\right)$ exceeding unity, the flow becomes super-critical, and the surface waves become so steep that they break intermittently and move upstream, although the sediment particles keep on moving downstream only [Fig. 4.1 (*f*)]. Sand Waves are then called **anti-dunes**, since the direction of movement of bed forms in this regime is opposite to that of the dunes. The sediment transport rate in this regime is obviously very high. The resistance to flow is, however, small compared to that of the ripple and dune regime. In case of canals and natural streams, anti-dunes rarely occur.

4.4. Mechanics of Sediment Transport

In the study of mechanics of sediment transport, we will throughout assume that the soil is *incoherent*. By incoherent soil, we mean that there are no cohesive forces between the particles, or in other words $c = 0$, such as in sands or gravels.

Most of our river beds are made up of sands and gravels, and hence, we confine ourselves to the mechanism of movement of such a soil only. Though cohesive clays, etc. are also sometimes met with, but no systematic study upon such soils has been undertaken and only very little work has been done in that direction.

By assuming the soil to be incoherent, each soil grain can be studied individually. The basic mechanism behind the phenomenon of sediment transport is the drag force exerted by water (or fluid) in the direction of flow, on the channel bed. This force, which is nothing but a pull of water on the wetted area, is known as *Tractive Force* or *Shear Force* or *Drag Force*.

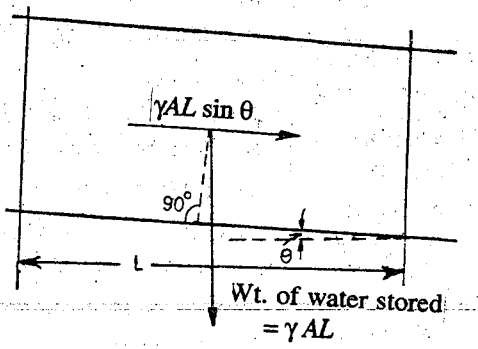


Fig. 4.2.

Let us consider a channel of length L and cross-sectional area A.

The volume of water stored in this channel reach $= AL$

Wt. of water stored $= \gamma_w AL$

where γ_w = unit wt. of water = $\rho_w g$, where ρ_w is the density of water.

Horizontal component of this wt. $= \gamma_w AL \sin\theta = \gamma_w ALS$

where S = channel bed slope.

This horizontal force exerted by water is nothing but Tractive force.

Average Tractive force per unit of wetted area

$$= \text{Unit Tractive Force } (\tau_0) = \frac{\gamma_w ALS}{\text{Wetted area}}$$

$$= \frac{\gamma_w ALS}{\text{Wetted perimeter} \times \text{Length}} = \frac{\gamma_w ALS}{P \cdot L} = \gamma_w \left(\frac{A}{P}\right) S. = \gamma_w R \cdot S \left(\because \frac{A}{P} = R \right)$$

where R = is the hydraulic mean depth of channel.

S = channel bed slope

γ_w = unit wt. of water

P = wetted perimeter

Hence, **Average Unit Tractive force,** also called Shear stress

$$= \tau_0 = \gamma_w RS \qquad ...(4.1)$$

It may be noted that the unit tractive force in channels, except for wide open channels, is not uniformly distributed along the wetted perimeter. A typical distribution of shear stress (unit-tractive force) on a trapezoidal channel section is shown in Fig. 4.3.

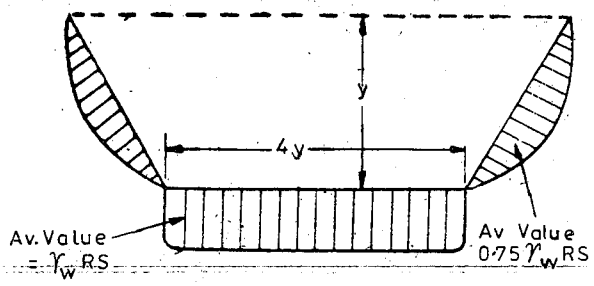


Fig. 4.3. Distribution of tractive force generated in a trapezoidal channel section.

Before entering into the mathematical aspect of sediment transport, we will again visualise the "threshold movement of the sediment", and its application for design of non-scouring channels.

4.4.1. Threshold Motion of the Sediment. When the velocity of flow through a channel is very small, the channel bed does not move at all, and the channel behaves as a **rigid boundary channel.** As the flow velocity increases steadily, a stage is reached when the shear force exerted by the flowing water on the bed particles will just exceed the force opposing their movement. At this stage, a few particles on the bed will just start moving intermittently. This condition is called the **incipient motion condition** or simply the **critical condition** or the **threshold point.**

A knowledge of the velocity at which such a critical condition occurs is quite helpful in designing stable non-scouring channels admitting clear waters, since this velocity will help us in fixing the hydraulic mean depth (R)* and bed slope (S_0) of the channel. The knowledge of this incipient motion condition is also required in some of the methods adopted for computation of sediment load.

* This will help in fixing depth (y) and bed width (b).

The experimental data on incipient motion condition was analysed using *critical tractive force approach,* for the first time by Shield, so as to help in designing stable channels in alluviums. This design basically assumes the entry of clean and clear water in the channel, which is designed to develop non-scouring highest possible flow velocity at the peak flow.

4.5. Shield's Entrainment Method for Design of Non-Scouring Stable Channels having Protected Side Slopes in Alluviums

Shield was the first investigator, who provided a semi-theoretical analysis of the problem of incipient condition of bed motion, and used it for designing non-scouring channels. He defined the **critical tractive stress** (τ_c) as that average shear stress (τ_0) acting on the bed of the channel, at which the sediment particle just begins to move. According to him, the bed particle begins to move when the drag force (F_1) exerted by the fluid on the particle, just equals or exceeds the resistance (F_2) offered by the particle to its movement.

(*i*) The drag force (F_1) exerted by the flow is given by :

$$F_1 = K_1 \left[C_D \cdot d^2 \cdot \frac{1}{2} \cdot \rho_w \cdot V_0^2 \right] \qquad ...(4.2)$$

where K_1 = *a* factor depending on the shape of the particle.

C_D = coefficient of drag.

d = The dia of the particle.

ρ_w = The density of the flowing fluid *i.e.* water.

V_0 = The velocity of flow at the top of the particle *i.e.* at the bottom of the channel.

Using *Karman-Prandtl equation* for the velocity distribution along a channel *x*-section, the velocity of flow at the bottom of the channel (V_0) can be expressed as :

$$\frac{V_0}{V^*} = f_1 \left(\frac{Vd}{\nu} \right) = f_1 \cdot R_e^* \qquad ...(4.3)$$

or

$$V_0 = V^* f_1 \cdot R_e^* \qquad ...(4.4)$$

where V^* = **Shear friction velocity** = $\sqrt{\frac{\tau_0}{\rho_w}}$

where τ_0 is the shear stress acting on the boundary of the channel.

ν = Kinematic viscosity of the flowing fluid *i.e.* water.

R_e^* = **Particle Reynold Number** = $V^* d/\nu$

Also, the coefficient of drag C_D is given by :

$$C_D = f' \cdot \left(\frac{V_0 \cdot d}{\nu} \right)$$

or $$C_D = f_2 \cdot \left(\frac{V^* d}{\nu}\right)$$

$$= f_2 \cdot R_e^* \qquad ...(4.5)$$

Substituting values of V_0 and C_D from Eqns. (4.4) and (4.5) in Eq. (4.2), we get

$$F_1 = K_1 \left[(f_2 \cdot R_e^*) \cdot d^2 \cdot \tfrac{1}{2} \rho_w \cdot (V^* f_1 \cdot R_e^{*2}) \right]$$

or $$F_1 = \left[K_1 \cdot f_2 \cdot f_1^2 \cdot \tfrac{1}{2} d^2 \cdot \rho_w \cdot V^{*2} R_e^{*2} \right] \qquad ...(4.6)$$

(*ii*) The particle resistance (F_2) is further given by :

$$F_2 = K_2 [d^3 \cdot (\rho_s - \rho_w)\, g] \qquad ...(4.7)$$

where ρ_s = density of particle

ρ_w = density of fluid or water.

S_s = Specific gravity of particle

γ_w = unit wt. of fluid or water.

K_2 = a factor dependent on the shape of the particle and internal friction of soil.

$$\therefore \quad F_2 = K_2 \left[d^3 \cdot \left(\frac{\rho_s}{\rho_w} - 1\right) \rho_w \cdot g \right] = K_2 [d^3 \cdot (S_s - 1) \cdot \gamma_w]$$

$$= K_2 \cdot \gamma_w d^3 (S_s - 1) \qquad ...(4.8)$$

At critical condition, equating Eqs. (4.6) and (4.8), and introducing subscript (c) to denote critical conditions, we get

$$K_1 f_2 \cdot f_1^2 \frac{1}{2} \cdot d^2 \cdot \rho_w \cdot V_{(c)}^{*2}\, R_{e(c)}^{*3} = K_2 \cdot \gamma_w d^3 \cdot (S_s - 1) \qquad ...(4.9)$$

or $$\frac{\rho_w \cdot V_{(c)}^{*2}}{\gamma_w \cdot d\,(S_s - 1)} = \left(\frac{2K_2}{K_1 f_2 f_1^2}\right) \cdot R_{e(c)}^{*-3}\,; \quad \text{But} \quad \rho_w \cdot V_{(c)}^{*2} = \tau_c$$

$$\therefore \quad \frac{\tau_c}{\gamma_w \cdot d \cdot (S_s - 1)} = F \cdot R_{e(c)}^* \qquad ...(4.14)$$

(some function of $R_{e(c)}$)

The left hand side term is a dimensionless number and is called the **Shield's Entrainment function,** and is usually denoted by F_s

$$\therefore \quad F_s = F \cdot (R^*_e) \qquad ...(4.10\ a)$$

at critical stage of bed movement in a channel in alluviums.

The above mathematical work shows that F_s *i.e.* $\dfrac{\tau_c}{\gamma_w \cdot d\,(S_s - 1)}$ is a function of R_e^* at critical stage of bed movement ; and based on the experimental work done by Shield, graphs have been plotted between F_s and R_e^*, as shown in Fig. 4.4. The obtained curve, forms a suitable basis for the design of channels, where it is required to prevent bed movement or to keep it to the minimum.

The application of this curve becomes more simple when Particle Reynold number is more than 400, and as such F_s becomes constant and equal to 0.056. Particle Reynold number, representing roughness, has been found to be more than 400, when the particle size exceeds 6 mm, such as for 'coarse alluvium soils'.

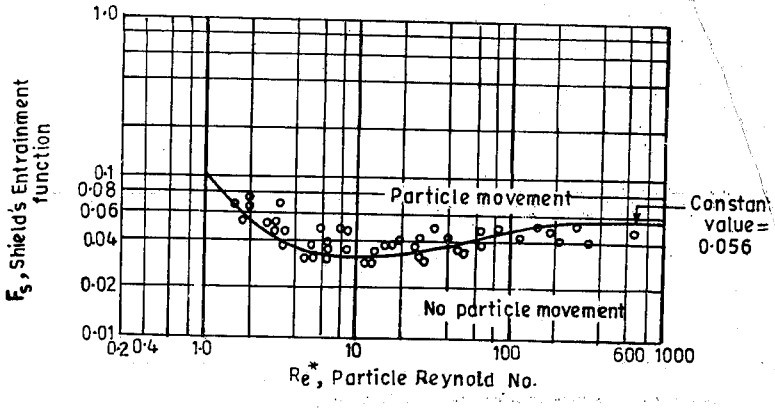


Fig. 4.4. Shield's curve for incipient motion condition.

Hence, for **designing non-scouring channels in coarse alluviums**

$$\frac{\tau_c}{\gamma_w d\,(S_s - 1)} = 0.056 \text{ (for } d > 6 \text{ mm)}^* \qquad \ldots(4.11)$$

where γ_w = unit wt. of water = 9.81 kN/m^3 or 1 t/m^3 or 1000 kgf/m^3.

The average shear stress caused on the bed of a channel by the flowing water is given by Eq. (4.1) as :

$$\tau_0 = \gamma_w RS$$

where, R = Hydraulic mean radius of the channel, *i.e.* A/P.
S = Bed slope.

Moreover, $\tau_0 \le \tau_c$;

$\therefore \quad \tau_0 \le \gamma_w d\,(S_s - 1)\,(0.056)$

or $\gamma_w RS \le \gamma_w d(S_s - 1)\,(0.056)$

or $RS \le d\,(S_s - 1)\,(0.056)$

or $RS \le d\,(2.65 - 1)\,(0.056)$

or $RS \le \dfrac{d}{11}$

or $d \ge 11\,RS$ ...(4.13)

Equation (4.13) gives the minimum size of the bed material or lining stone that will remain at rest in a channel of given R and S.

Since with the passage of time, the channel bed becomes Armored (*i.e.* the smaller stones are flushed out of the surface lining of the coarser stones), actual size of bed or lining to be used should be somewhat more than what is calculated from the equation $d = 11\,RS$.

*Mittal and Swamee has worked out a general relation between τ_c and d which gives results within +5% of the values given by Shield's curve, for all values of d. The relation for water and soil of $S_s = 2.65$, is given by equation

$$\tau_c\,(\text{N/m}^2) = 0.155 + \frac{0.409 d^2_{mm}}{\sqrt{1 + 0.177 d^2_{mm}}} \qquad \ldots(4.12)$$

Example 4.1. *An irrigation channel is to be constructed in coarse alluvium gravel with D-75 size of 5 cm. The channel has to carry 3 cumecs of discharge and the longitudinal slope is 0.01. The banks of the channel will be protected by grass against scouring. Find the minimum width of the channel.*

Solution. d = grain dia = 5 cm = 0.05 m (> 6 mm)

By **Strickler's formula***, we know that Manning's rugosity coefficient (n) is given as:

$$n = \frac{1}{24} d^{1/6} \qquad \text{where } d \text{ is in metres}$$

$$= \frac{1}{24} \times \left(\frac{5}{100}\right)^{1/6} = \frac{1}{24} (0.05)^{0.16} = \frac{1}{24} \times 0.619 = 0.0258 \text{ m}.$$

Now, $d \geq 11\, RS$, or $R \leq \frac{d}{11\, S} = \left(\frac{5}{100} \times \frac{1}{11 \times 0.01}\right) = 0.455$ m,

or $R_{max} = 0.455$ m

Now using $V = \frac{1}{n} R^{2/3} S^{1/2}$ (Manning's formula), we have

$$V_{max} = \frac{1}{0.0258} (0.455)^{2/3} (0.01)^{1/2}$$

$$= \frac{1}{0.0258} (0.592)\, (0.1) = 2.29 \text{ m/sec}.$$

Assuming $R = y$ (1st app.), $Q = AV = by \times V = b \cdot RV$

$Q = b \cdot RV$, if R and V are taken maximum b will be minimum.

$$\therefore \qquad 3 = b_{min} \times 0.455 \times 2.29$$

or $$b_{min} = \frac{3}{0.455 \times 2.29} = 2.88 \text{ m}.$$

Use a conservative value of base width as 3 m. **Ans.**

Example 4.2. *Water flows at a depth of 0.6 m in a wide stream having a bed slope of 1 in 2500. The median diameter of the sand bed is 1.0 mm. Determine whether the soil grains are stationary or moving, and comment as to whether the stream bed is scouring or non-scouring.*

Solution. Since the given size of bed particles is 1.0 mm, which is less than 6 mm, we can not use Shield's Eq. (4.11), since R_e^* in this case will be less than 400.

We will, therefore, use the general Eq. (4.12), which is valid for all sizes of d.

$$\tau_c \text{ (N/m}^2\text{)} = 0.155 + \frac{0.409\, d^2_{mm}}{\sqrt{1 + 0.177\, d^2_{mm}}}$$

$$= 0.155 + \frac{0.409 \times 1}{\sqrt{1 + 0.177 \times 1}} = \mathbf{0.53\ N/m^2}.$$

---

*Strictly speaking, the **Strickler's formula** is applicable to the *rigid boundary channels* only and not to the *moveable boundary channels*, since it gives the roughness coefficient (n') due to grain roughness alone, and does not account for form roughness, which is caused by the undulations in the bed of a moveable boundary channel. The true value of n in Manning's equation will, on the other hand, represent roughness of bed consisting of grain roughness as well as the form roughness, together. This formula is therefore valid for rivers with beds of coarse materials, practically free from ripples.

Also using Eq. (4.1), we have

$$\tau_0 = \gamma_w \cdot R \cdot S = 9.81 \times 0.6 \times \frac{1}{2500} \text{ kN/m}^2$$

$$= 2.35 \times 10^{-3} \text{ kN/m}^2 \qquad \left(\gamma_w = 9.81 \text{ kN/m}^3 \text{ for water}\right.$$

$$= 2.35 \text{ N/m}^2 \qquad \left.\because\ R \approx y \text{ for wide streams}\right)$$

$$= 2.35 \text{ N/m}^2\text{; which is more than } 0.53 \text{ N/m}^2.$$

Since $\tau_0 > \tau_c$, the soil grains will not be stationary, and the scouring and sediment transport will occur. **Ans.**

4.6. Stability of Channel Slopes (Design of Non-Scouring Channels with Unprotected Side Slopes)

Upto now, we have considered the stability of horizontal beds, where the shear stress τ_0 (given by $\tau_0 = \gamma_w RS$) was the only disturbing force. But on side slopes of channels, one more disturbing force, *i.e.*, the component of the weight of the particle, also comes into picture.

We will now consider a grain on the side slope of a channel. Various forces acting on this grain are shown in Fig. 4.5 (*a*).

Now, let τ_c' be the shear stress required to move the grain of weight W on the side slopes. The free-body diagram of various forces acting on the grain is shown in Fig. 4.5 (*c*).

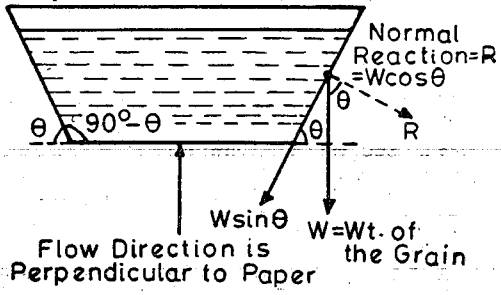
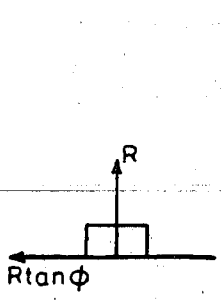
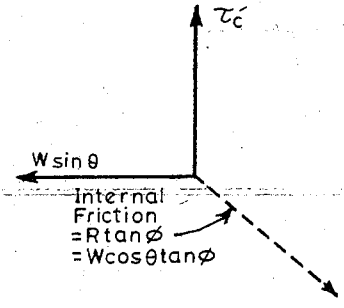


Fig. 4.5. (*a*) Forces acting on a grain on the side slope of a channel.



(*b*) where ϕ = Angle of repose of soil.



(*c*) Free body diagram of forces.

Fig. 4.5.

Let τ_e represents the critical shear stress or the shear stress required to move a similar grain on a horizontal bed, as shown in Fig. 4.6.

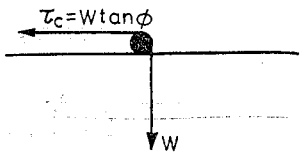


Fig. 4.6.

Now, $\tau_c = W \tan \phi$

From Fig. 4.5 (*b*),

$$(\tau_c')^2 + (W \sin \theta)^2 = [(W \cos \theta) \tan \phi]^2$$

or $$\tau_c'^2 + \left[\frac{\tau_c}{\tan \phi} \sin \theta\right]^2 = \left[\frac{\tau_c}{\tan \phi} \cos \theta \tan \phi\right]^2$$

or $$r_c'^2 + \frac{\tau_c^2}{\tan^2 \phi} \sin^2 \theta = \tau_c^2 \cos^2 \theta$$

or $$\tau'^2_c = \tau_c^2 \left[\cos^2 \theta - \frac{\sin^2 \theta}{\tan^2 \phi}\right] \quad ...(4.14)$$

or $$\frac{\tau_c'^2}{\tau_c^2} = \cos^2 \theta - \frac{\sin^2 \theta}{\tan^2 \phi} = \cos^2 \theta \left[1 - \frac{\tan^2 \theta}{\tan^2 \phi}\right]$$

or $$\frac{\tau_c'}{\tau_c} = \cos \theta \sqrt{1 - \frac{\tan^2 \theta}{\tan^2 \phi}} \quad ...(4.15)$$

Equation (4.14) can also be written as

$$\left(\frac{\tau_c'}{\tau_c}\right)^2 = \left[\cos^2 \theta + (\sin^2 \theta - \sin^2 \theta) - \frac{\sin^2 \theta}{\tan^2 \phi}\right] = \left[1 - \sin^2 \theta - \frac{\sin^2 \theta}{\tan^2 \phi}\right]$$

$$= \left[1 - \sin^2 \theta \left(1 + \frac{1}{\tan^2 \phi}\right)\right] = \left[1 - \sin^2 \theta \times \frac{1 + \tan^2 \phi}{\tan^2 \phi}\right]$$

$$= \left[1 - \sin^2 \theta \cdot \frac{\sec^2 \phi}{\tan^2 \phi}\right] = \left[1 - \frac{\sin^2 \theta}{\sin^2 \phi}\right]$$

or $$\boxed{\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}} \quad ...(4.16)$$

The above equation shows that $\tau_c' < \tau_c$: which means *that the shear stress required to move a grain on the side slopes is less than the shear-stress required to move the grain on canal bed.*

Moreover, on the channel bed, the average value of actual shear stress generated by the flowing water in a channel of given R and S, is given by equation (4.1), as

$$\tau_0 = \gamma_w RS$$

while on slopes, this value is given by $(\tau_0') = 0.75 \gamma_w RS$ ...(4.17)

The above distribution of shear-stresses was shown earlier in Fig. 4.3.

Example 4.3. *A canal is to be designed to carry a discharge of 56 cumec. The slope of the canal is 1 in 1000. The soil is coarse alluvium having a grain size of 5 cm. Assuming the canal to be unlimited and of a trapezoidal section, determine a suitable section for the canal, ϕ may be taken as 37°.*

Solution. First of all, let us choose suitable side slopes, such that $\theta < \phi$. Let θ be 30°.

Now $$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{\sin^2 30°}{\sin^2 37°}} = \sqrt{1 - \left(\frac{0.50}{0.62}\right)^2} = 0.557$$

$$\therefore \quad \frac{\tau_c'}{\tau_c} = 0.557$$

Therefore, minimum shear stress required to dislodge the grain on side slope is given by

$$\tau_c' = 0.557\,\tau_c.$$

Hence, for stability, the shear stress actually going to be generated on the slopes of a channel of given R and S must be less than or equal to $0.557\,\tau_c$

i.e., $\tau_0' \le 0.557\,\tau_c$ (for stability)

But the shear stress actually going to be generated on the side slopes of a channel of given R and S, from Eq. (4.17), is

$$= \tau_0' = 0.75\,\gamma_w RS$$

$$0.75\,\gamma_w RS \le 0.557\,\tau_c$$

But $$\tau_c = \gamma_w RS = \frac{\gamma_w \cdot d}{11}$$

$$\therefore\ 0.75\,\gamma_w RS \le 0.557 \cdot \frac{\gamma_w d}{11}$$

or $$RS \le \frac{0.557}{0.75 \times 11} d$$

or $$RS \le 0.0676\,d$$

or $$RS \le (0.0676 \times 0.05)\ \text{m}.$$

or $$R \times \frac{1}{1000} \le (0.0676 \times 0.05) \quad \text{when } R \text{ is in metres}$$

or $$R \le 3.38\ \text{m}$$

$$\therefore \quad y \le 3.38\ \text{m}. \quad (\because\ y \approx R)$$

With 20% factor of safety, use $y = 2.8$ m.

Hence, choose the depth as 2.8 m. Let us now choose the base width b in such a way as to have a discharge of 56 cumec.

Let us use hit and trial method to determine b.

$$A = y\,(b + x) = 2.8\,(b + 4.85)$$

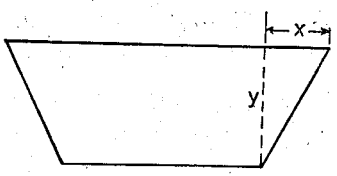
and $$P = b + 2\sqrt{x^2 + y^2} \quad \text{(w.r. to Fig. 4.7)}$$

$$= b + 2\sqrt{23.6 + 7.84} = b + 11.22$$

$$n = \frac{1}{24} d^{1/6} = \frac{1}{24} \times (0.05)^{1/6} = 0.0258$$

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2}$$

$$= \frac{1}{0.0258} R^{2/3} \sqrt{\frac{1}{1000}} = 1.223\,R^{2/3}.$$



$$\frac{y}{x} = \tan 30° = \frac{1}{\sqrt{3}}$$

or $$x = 2.8\sqrt{3} = 4.85\ \text{m}$$

Fig. 4.7.

Choose a number of trial values of b and assuming a depth of 2.8 m, proceed by the given table 4.1, till a discharge of 56 cumec is reached at $b = 6.3$ m.

Table 4.1

b in metres	$A = 2.8 \times (b + 4.85)$ m^2	P *in* $m = (b + 11.22)$	$R = \frac{A}{P}$ m	$R^{2/3}$	$1.223\ R^{2/3} = V$ *m/sec*	$Q = A \times V$ *cumecs*
3	22	14.22	1.544	1.337	1.635	36
4	24.8	15.22	1.625	1.382	1.695	42
5	27.6	16.22	1.695	1.422	1.745	48.3
6	30.4	17.22	1.764	1.470	1.800	54.8
6.3	31.22	17.52	1.782	1.470	1.798	**56.1 ; say 56**

Hence, use 6.3 m base width and 2.8 m depth **Ans.**

Example 4.4. *A trapezoidal channel with side slopes 1.5 horizontal : 1 vertical is required to carry 15 m^3/s of flow with a bed slope of 1 in 4000. If the channel is lined, the Manning's coefficient n will be 0.014, and it will be 0.028 if the channel is unlined. Calculate the average boundary shear stress if a hydraulically efficient lined channel is adopted. What percentage of earthwork is saved in a lined section, relative to an unlined section, when hydraulically efficient section is used in both the cases ? The free board can be assumed to be 0.75 m in both the cases and the lining can be assumed to be up to the top of the section.* *(Civil services, 1991)*

Solution. $Q = 15\ m^3/s$ (given)

$N = 0.014$ for lined section, and
0.028 for unlined section (given)

$S = \frac{1}{4000}$ (given)

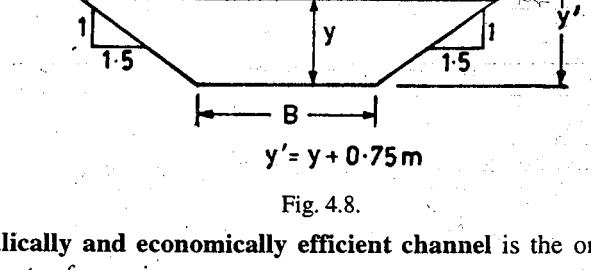


Fig. 4.8.

A hydraulically and economically efficient channel is the one *which possesses minimum perimeter for a given area ;*

For a trapezoidal channel having 1.5 H : 1V side slopes, bed width B, and depth y, we have

$$P = B + 2\,[\text{sloping side}] = B + 2\,[1^2 + 1.5^2]\,y = B + 2 \times 1.8y$$

or $$P = B + 3.6y \qquad ...(i)$$

Also $$A = [B + y \times 1.5]\,y = (B + 1.5y)\,y$$

or $$\frac{A}{y} = B + 1.5y$$

or $$B = \frac{A}{y} - 1.5y \qquad ...(ii)$$

Substituting in (*i*), we get

$$P = \left[\frac{A}{y} - 1.5y\right] + 3.6y$$

or $$P = \frac{A}{y} + 2.1y \qquad ...(iii)$$

For minimum perimeter (with constant A), $\frac{dP}{dy} = 0$

$\therefore$ $$\frac{dP}{dy} = A(-1y^{-2}) + 2.1 \times 1 = -\frac{A}{y^2} + 2.1 = 0$$

or $$\frac{A}{y^2} = 2.1 \qquad \text{or} \qquad \mathbf{A = 2.1\, y^2}$$

Substituting in (*iii*), we get

$$P = \frac{2.1y^2}{y} + 2.1y = 4.2y$$

$\therefore$ $$R = \frac{A}{P} = \frac{2.1y^2}{4.2y} = \mathbf{\frac{y}{2}}$$

For a channel having 1.5 H : 1 V slopes, therefore, the most efficient section will have $A = 2.1y^2$, and $R = \frac{y}{2}$.

Now, using $Q = \frac{1}{n} A \cdot R^{2/3} S^{1/2}$, we have

$$15 = \frac{1}{0.014} \times (2.1\, y_1^2) \left(\frac{y_1}{2}\right)^{2/3} \frac{1}{\sqrt{4000}}$$

or $$y_1 = (10.039)^{3/8} \qquad \text{or} \qquad y_1 = \mathbf{2.37\ m}$$

$\therefore$ $B_1 = 0.6 \times 2.37 = \mathbf{1.42\ m}$ $\therefore$ $R_1 = \frac{y_1}{2} = 1.19$ m

Now, **average boundary shear stress** τ_0 *is given by eqn. (4.1) as :*

$$\tau_0 = \gamma_w \cdot RS$$

where γ_w = unit wt. of water = 9.81 kN/m^3

$R = 1.19$ m

$$S = \frac{1}{4000}$$

$\therefore$ $$\tau_0 = 9.81 \times 1.19 \times \frac{1}{4000}$$

$$= 2.918 \times 10^{-3}\ \text{kN/m}^2 = \mathbf{2.918\ N/m^2}. \quad \textbf{Ans.}$$

Now, we compute X-sectional areas of excavation in both cases :

Using free-board of 0.75 m

(for lined section) $= A_1 = (B_1 + 1.5\, y'_1)\, y'_1$

where $y_1' = y_1 + 0.75 = 2.37 + 0.75 = 3.12$ m

or $$A_1 = (1.42 + 1.5 \times 3.12)\, 3.12\ \text{m}^2 = \mathbf{19.04\ m^2} \qquad ...(i)$$

For unlined section of bed width B_2 and depth y_2, we have

$$n = 0.028$$

$$\therefore \quad 15 = \frac{1}{0.028}(2.1\, y_2^2)\left(\frac{y_2}{2}\right)^{2/3} \cdot \frac{1}{\sqrt{4000}}$$

or $$y_2 = (2 \times 10.039)^{3/8} = 3.08 \text{ m}$$

$$\therefore \quad B_2 = 0.6 y_2 = 0.6 \times 3.08 = 1.85 \text{ m}$$

$$A_2 = (B_2 + 1.5\, y_2')\, y_2' \qquad \text{where } y_2' = 3.08 + 0.75 = 3.83 \text{ m}$$

$$= (1.85 + 1.5 \times 3.83)\, 3.83 \text{ m}^2 = \mathbf{29.09\ m^2} \qquad ...(ii)$$

Percentage saving in earth work due to lining

$$= \frac{A_2 - A_1}{A_2} \times 100 = \frac{29.09 - 19.04}{29.09} \times 100 = \mathbf{34.55\%\ Ans.}$$

Example 4.5 *A most efficient trapezoidal section is required to give a maximum discharge of 21.5 m^3/s of water. The slope of the channel bottom is 1 in 2500, Taking $C = 70\ m^{1/2}/s$ in Chezy's equation, determine the dimensions of the channel. Also determine the value of Manning's 'n', taking the value of velocity of flow as obtained for the channel by Chezy's equation.* (*Engg Services 1997*)

Solution. We shall first derive the equations to be used in the question as follows.

For the most efficient channel, the wetted perimeter must be minimum for a given area. Thus, for a trapezoidal channel of bed width B, depth y, and side slopes $m : 1$ ($H : V$), we have

$$A = (B + m \cdot y)\, y \qquad ...(i)$$

$$P = 2 \cdot \sqrt{(1 + m^2)}\, y + B \qquad ...(ii)$$

or $$P = 2\sqrt{(1 + m^2)} \cdot y + \left(\frac{A}{y} - m \cdot y\right)$$

or $$P = \frac{A}{y} - m \cdot y + 2\,(1 + m^2)^{1/2} \cdot y$$

or $$\frac{dP}{dy} = -A \cdot y^{-2} - m + 2\sqrt{1 + m^2} = 0$$

or $$\frac{A}{y^2} = 2\sqrt{1 + m^2} - m$$

or $$\boxed{A = [2\sqrt{1 + m^2} - m]\, y^2} \qquad ...(iii)$$

But from (*i*) $$B = \frac{A}{y} - my = [2\sqrt{1 + m^2} - m]\, y - my$$

or $$B = 2\,[\sqrt{1 + m^2} - m]\, y \qquad ...(iv)$$

Note : The second derivative of P with respect to y is worked out to be $2A/y^3$, which is + ve, and hence the condition obtained above is for minimum P.

Using the above worked out relation, we can write

$$P = 2\sqrt{1 + m^2} \cdot y + B = 2\sqrt{1 + m^2} \cdot y + 2\,[\sqrt{1 + m^2} - m]\, y$$

$$= 4\sqrt{1 + m^2} \cdot y - 2my$$

or $$P = 2 \cdot y\,[2\sqrt{1 + m^2} - m]$$

$$\therefore \quad R = \frac{A}{P} = \frac{[2\sqrt{1+m^2} - m]\, y^2}{[2\sqrt{1+m^2} - m]\, 2y} = \frac{y}{2}$$

or $$\boxed{R = \frac{y}{2}} \qquad \ldots(1)$$

In the expression for P, there are two variables y and n. A second condition for min. P can be obtained by equating $\frac{dP}{dm} = 0$, holding y as constant.

$\therefore$ using $P = 2y\left[2\sqrt{1+m^2} - m\right]$

or $$P = 4 \cdot y\sqrt{1+m^2} - 2y \cdot m$$

or $$\frac{dP}{dm} = 4 \cdot y \frac{1}{2}(1+m^2)^{-1/2} \cdot 2m - 2y \times 1 \qquad \text{(with constant } y\text{)}$$

$$= \frac{4y \cdot m}{\sqrt{1+m^2}} - 2y = 0$$

or $\frac{4ym}{\sqrt{1+m^2}} = 2y$ or $\frac{2m}{\sqrt{1+m^2}} = 1$

or $\sqrt{1+m^2} = 2m$ or $(1+m^2) = 4m^2$ or $3m^2 = 1$

or $$m = \frac{1}{\sqrt{3}} \qquad \ldots(2)$$

Hence, for the most efficient trapezoidal channel, side slopes should be $\frac{1}{\sqrt{3}} : 1$ (H : V)

Using the above two conditions, **for the most efficient channel** ; *i.e.* $m = \frac{1}{\sqrt{3}}$ and $R = \frac{y}{2}$, we can solve the given question as follows :

Using $Q = C \cdot \sqrt{R \cdot S} \times A$, we have

$$Q = 21.5\ \text{m}^3/\text{s},$$
$$C = 70\sqrt{\text{m}}/\text{sec}$$
$$S = \frac{1}{2500}$$

$\therefore$ $$21.5 = 70 \times \sqrt{R} \cdot \frac{1}{\sqrt{2500}} \times A$$

or $$21.5 = \frac{70}{50}\sqrt{R} \cdot A$$

For the most efficient trepezoidal channel, we have side slopes $m : 1$ ($H : V$), where $m = \frac{1}{\sqrt{3}}$; and $R = \frac{y}{2}$.

Also, $A = [2\sqrt{1+m^2} - m]\, y^2$ ..... from Eq. (*iii*)

$$= \left[2\sqrt{1+\frac{1}{3}} - \frac{1}{\sqrt{3}}\right] y^2 = (2.3094 - 0.5773)y^2 = 1.732y^2$$

$$\therefore \qquad 21.5 = \frac{70}{50} \cdot \sqrt{\frac{y}{2}} \times 1.732 y^2$$

or $$\frac{21.5 \times 50 \times \sqrt{2}}{70 \times 1.732} = y^{2.5} \qquad \text{or} \qquad \mathbf{y = 2.75\ m} \qquad \textbf{Ans.}$$

Also $$A = 1.732 \cdot (2.75)^2 = 13.10$$

But $$A = (B + m \cdot y)\, y = \left(B + \frac{1}{\sqrt{3}} \times 2.75\right) 2.75$$

$\therefore$ $$13.10 = (B + 0.577 \times 2.75)\, 2.75 \quad \text{or} \quad 13.10 = (B + 1.588)\, 2.75$$

or $$\mathbf{B = 3.18\ m}$$

The channel dimensions are thus worked out as :

$$B = 3.18\ \text{m} \qquad \text{and} \qquad y = 2.75\ \text{m}$$

Side slopes $= \frac{1}{\sqrt{3}} : 1$ (*i.e.* sides inclined at 60° to horizontal)

Velocity as per Chezy's Equation determined above is

$$V = C \cdot \sqrt{RS} = 70 \cdot \sqrt{R} \cdot \frac{1}{50} = 70 \cdot \sqrt{\frac{y}{2}} \cdot \frac{1}{50}$$

$$= 70 \times \sqrt{\frac{2.75}{2}} \cdot \frac{1}{50} = 1.64\ \text{m/s}$$

With Manning's equation

$$V = \frac{1}{n} \cdot R^{2/3} \cdot \sqrt{S} = \frac{1}{n} \cdot \left(\frac{y}{2}\right)^{2/3} \cdot \sqrt{S} = \frac{1}{n} \times \left(\frac{2.75}{2}\right)^{2/3} \cdot \frac{1}{50}$$

or $$1.64 = \frac{1}{n} \cdot (1.375)^{2/3} \cdot \frac{1}{50}$$

or $$n = \frac{(1.375)^{2/3}}{50 \times 1.64} \qquad \text{or} \qquad n = 0.015 \quad \textbf{Ans.}$$

4.7. Design of Stable Channels in India

So long as the average shear stress (τ_0) acting on the boundary of an alluvial channel is less than the critical shear stress (τ_c), the channel shape remains unchanged, and hence the channel can be considered of rigid boundary. The resistance equations, such as those given by Chezy's formula and Manning's formula, remain well applicable to such channels. However, as soon as the sediment movement starts, undulations develop on the bed, which increases the boundary resistance of the channel. Besides this, some energy is required to move the grains. The suspended load, carried due to turbulence in the flow, further affects the resistance of the alluvial streams. All these factors render the evaluation of resistance of alluvial streams to be a very complex problem, and the complexity further increases if one includes the effects of the channel shape, non-uniformity of sediment size, discharge variation, and other such factors. None of the resistance equations developed so far, takes all these factors into account. The direct accurate mathematical solution to the design of channels in alluvial soils is, therefore, not an easy job ; and hence in India, alluvial channels are designed on the basis of hypothetical theories given by Kennedy and Lacey. These theories are based on experiments and experience gained on the existing channels over the past many years. The semi-theoretical approach discussed in the previous article, is however, used to design channels in countries like America and Europe.

4.7.1. Problem in India. In prehistoric periods, the area bounded by Indo-Gangetic plain, *i.e.* the area starting from Himalayas to Vindhya mountains, used to be in the form of depressions with water flowing over it. But with the passage of time, it was filled up with loosely filled fine silt particles, thereby forming nothing, but what is known as **alluvial soil.** Almost all the north Indian rivers flow through such soils and, therefore, do carry a certain amount of sediment. Artificial channels have to carry their water supply from such rivers, and thus carrying sediment.

We also know that water moving with a given velocity and a certain depth can carry in suspension, only a certain amount of silt of a certain nature. If water of a given velocity and depth is not fully charged with silt (that it can carry in suspension) it will scour the bed and sides of the channel, till it is fully charged with silt. *Hence, if the velocity of flow in the channel is more, the bed and the banks are likely to be eroded, and similarly, if the velocity is less, the silt which was formerly carried in suspension is likely to be dropped.*

Silting and scouring in channels is not very uncommon and is understandable, but must be avoided by proper designs. Scouring lowers the full supply level and causes loss of command. It may also cause breaching of canal banks and failure of foundations of irrigation structures. Silting interferes with the proper working of a channel, as the channel section gets reduced by siltation, thereby reducing the discharging capacity of the channel.

Therefore, while thinking to design a properly functioning channel, one must think to design such a channel in which neither silting nor scouring takes place. Such channels are known as stable channels or regime channels.

4.7.2. Regime Channels. A channel is said to be in a state of 'Regime', if the flow is such that 'silting and scouring' need no special attention. Such a state is not easily possible in rivers, but in artificial channels, such a state can be obtained by properly designing the channel.

The basis for designing such an ideal, non-silting, non-scouring channel is that, whatever silt has entered the channel at its head is kept in suspension, so that it does not settle down and deposit at any point of the channel. Moreover, the velocity of the water should be such that it does not produce local silt by erosion of channel bed and slopes.

4.7.3. Kennedy's Theory (1895). R.G. Kennedy, an Executive Engineer of Punjab P.W.D, carried out extensive investigations on some of the canal reaches in the upper Bari Doab Canal System. He selected some straight reaches of the canal section, which had not posed any silting and scouring problems during the previous 30 years or so.

From the observations, he concluded that the silt supporting power in a channel cross-section was mainly dependent upon the generation of the eddies, rising to the surface. These eddies are generated due to the friction of the flowing water with the channel surface. The vertical component of these eddies try to move the sediment up, while the weight of the sediment tries to bring it down, thus keeping the sediment in suspension. So if the velocity is sufficient to generate these eddies, so as to keep the sediment just in suspension, silting will be avoided. Based upon this concept, he defined the **critical velocity** (V_0) in a channel as the *mean velocity* (across the section) which will just keep the channel free from silting or scouring, and related it to the depth of flow by the equation

$$V_0 = c_1 \cdot y^{c_2}$$

where c_1 and c_2 are constants depending upon silt charge.

c_1 and c_2 were found to be 0.55 and 0.64 (in M.K.S. or S.I. units), respectively.

Therefore, $V_0 = 0.55\, y^{0.64}$ ...(4.18)

Since this formula was worked out especially for the upper Bari Doab canal system, it could not have been applicable in toto to other canals or canal systems due to variation in the type of soil (or silt) at various canal sites. Realising this lacuna, Kennedy later introduced a factor (m) in this equation, to account for the type of soil through which the canal was to pass. This factor, which was dependent upon the silt grade, was named as **critical velocity ratio (C.V.R.)** and denoted by m.

The equation for critical velocity was, thus, modified as :

$$\boxed{V_0 = 0.55\, m y^{0.64}} \quad ...(4.19)$$

where V_0 = Critical velocity in the channel in m/s.

y = water depth in channel in m

$m = C.V.R$

For sands coarser than the standard, the values of m were given from 1.0 to 1.2 ; and for sands finer than the standard, m was valued between 1.0 to 0.7, as shown in Table 4.2.

Table 4.2. Recommended Values of C.V.R. (m)

S.No.	*Type of silt*	*Value of m*
1.	Silt of River Indus (Pakistan)	0.7
2.	Light sandy silt in North Indian Rivers	1.0
3.	Light sandy silt, a little coarser	1.1
4.	Sandy, loamy silt	1.2
5.	Debris of hard soil	1.3

Design procedure. Determine the critical velocity V_0 by the above Eq. (4.19) by assuming a trial depth, and then determine area by dividing discharge by velocity. Then determine channel dimensions. Finally, compute the actual mean velocity (V) that will prevail in the channel of this cross-section, by using Kutter's formula, Manning's formula, etc. If the two velocities V_0 and V work out to be the same, then the assumed depth is all right, otherwise change it and repeat the procedure, till V and V_0 become equal.

Kutter's Formula

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS} \quad ...(4.20)$$

Manning's Formula

$$V = \frac{1}{n} R^{2/3} \cdot S^{1/2} \quad ...(4.21)$$

where V = Velocity of flow in metres/sec.

R = Hydraulic mean depth in metres.

S = Bed slope of the channel.

n = Rugosity coefficient.

The values of n in both these equations depend upon channel condition and also upon discharge. The values of n may be taken as given in Table 4.3.

Table 4.3. Recommended Values of Manning's Coefficient n for Unlined Channels

Condition of channel	*Value of n*
Very good	0.0225
Good	0.025
Indifferent	0.0275
Poor	0.030

The Central Board of Irrigation and Power (India) has recommended the following values of n for different discharges.

Table 4.4. Values of Manning's n for Different Discharges

Discharge in cumec	*Value of n for Unlined Channels*
14 to 140	0.025
140 to 280	0.0225
280 and above	0.020

Chezy's Formula

$$V = C\sqrt{RS} \qquad ...(4.22)$$

where C = a constant depending upon the shape and surface of the channel.

R and S have the same meaning as in eq. (4.21).

The actual mean velocity(V) generated in the channel can be computed by any of these three resistance equations, but generally Kutter's equation is used with Kennedy's theory.

Example 4.6. *Design an irrigation channel to carry 50 cumecs of discharge. The channel is to be laid at a slope of 1 in 4000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.023.*

Solution. $Q = 50$ cumecs, $S = \dfrac{1}{4000}$

$m = 1.1$, $n = 0.023$

Use equation (4.19), as, $V_0 = 0.55m \cdot y^{0.64}$

Assume a depth equal to 2 m

$$V_0 = 0.55 \times 1.1 \times (2)^{0.64} = 0.605 \times 1.558 = \mathbf{0.942}\ \text{m/sec}$$

$$A = \frac{Q}{V_0} = \frac{50}{0.942} = 53.1\ \text{m}^2.$$

Assume side slopes as $\frac{1}{2}:1\left(\frac{1}{2}H:1V\right)$

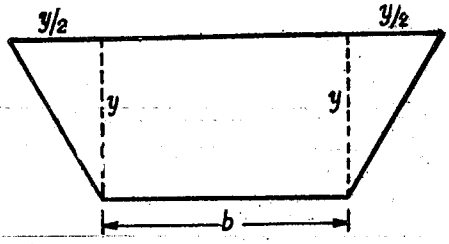


Fig. 4.9.

Now, $A = y\left(b + y\cdot\frac{1}{2}\right)$

$\therefore \quad 53.1 = 2\,(b+1)$

or $26.55 = b + 1$

or $b = 25.55$ m

and $P = b + 2\sqrt{\left(1+\frac{1}{4}\right)}\times y$

or $P = b + 2\,\frac{\sqrt{5}}{2}\,y = 25.55 + \sqrt{5}\times 2 = 30.03$

$$R = \frac{A}{P} = \frac{53.1}{30.03} = 1.77 \text{ m.}$$

But, from eqn. (4.20),

$$V = \left[\frac{\frac{1}{n} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{n}{\sqrt{R}}}\right]\sqrt{RS}$$

$$\therefore \quad V = \left[\frac{\frac{1}{0.023} + 23 + \frac{0.00155}{1/4000}}{1 + \left(23 + \frac{0.00155}{1/4000}\right)\frac{0.023}{\sqrt{1.77}}}\right]\sqrt{1.77\times\frac{1}{4000}}$$

$$= \left[\frac{43.5 + (23 + 6.2)}{1 + \frac{29.2\times 0.023}{1.33}}\right]\left[1.33\times\frac{1}{63.3}\right]$$

$$= \frac{72.7}{1+0.505}\times 1.33\times\frac{1}{63.3} = \frac{72.7}{1.505}\times 1.33\times\frac{1}{63.3}$$

$$= 1.016 \text{ m/sec} > 0.942 \text{ ; or } V > V_0.$$

In order to *increase* the critical velocity (V_o),, we have to *increase* the depth. So increase the depth.

Use 3 m depth :

$$V_0 = 0.605\times(3)^{0.64} = 0.605\times 2.02 = 1.22 \text{ m/sec.}$$

$$A = \frac{50}{1.22} = 40.8 \text{ m}^2.$$

$$40.8 = 3\,(b + \tfrac{1}{2}\cdot 3)$$

or $13.6 - 1.5 = b = 12.1$ m.

$$P = 12.1 + 2\times\frac{\sqrt{5}}{2}\,3 = 12.1 + 6.72 = 18.82$$

$$R = \frac{A}{P} = \frac{40.8}{18.82} = 2.17 \text{ ; therefore } \sqrt{R} = 1.47.$$

$$V = \frac{43.5 + 29.2}{1 + \dfrac{29.2 \times 0.023}{1.47}} + \left[1.47 \times \frac{1}{63.3}\right] = \frac{72.7}{1.45} \times 1.47 \times \frac{1}{63.3}$$

$= 1.16$ m/sec. < 1.22 ; or $V < V_0$

So reduce the depth.

Use 2.5 m depth

$$V_0 = 0.605 \times (2.5)^{0.64} = 0.605 \times 1.797 = 1.087 \text{ m/sec.}$$

$$A = \frac{50}{1.087} = 46$$

$$46 = 2.5\,(b + \tfrac{1}{2} \cdot 2.5)$$

$$18.4 - 1.25 = b = 17.15 \text{ m}$$

$$P = 17.15 + \sqrt{5} \times 2.5 = 17.15 + 5.58 = 22.73$$

$$R = \frac{A}{P} = \frac{4}{22.73} = 2.02 \text{ ; therefore } \sqrt{R} = 1.42$$

$$V = \frac{72.7}{1 + \dfrac{29.2 \times 0.023}{1.42}} (1.42) \left(\frac{1}{63.3}\right) = \frac{72.7}{1.472} \times \frac{1.42}{63.3}$$

$= 1.1$ m/sec > 1.087 ; $V > V_0$

So increase the depth.

Use 2.7 m depth

$$V_0 = 0.605 \times 1.189 = 1.147$$

$$A = \frac{50}{1.147} = 43.5$$

$$43.5 = 2.8\,(b + \tfrac{1}{2} \cdot 2.8)$$

$$15.54 - 1.4 = b = 14.14 \text{ m}$$

$$P = 14.14 + \sqrt{5} \times 2.8 = 14.14 + 6.26 = 20.40$$

$$R = \frac{43.5}{20.4} = 2.13, \text{ therefore, } \sqrt{R} = 1.46$$

$$\therefore \quad V = \left[\frac{72.7}{1 + \dfrac{29.2 \times 0.023}{1.46}}\right]\left[\frac{1.46}{63.3}\right] = \left[\frac{72.6}{1.46}\right]\left[\frac{1.46}{68.3}\right]$$

$= 1.148$ m/sec ≈ 1.147 or $V \approx V_0$.

Actual velocity V tallies with V_0.

Hence, use the depth equal to 2.7 m and base width 14.14 m. (say 14.2 m) with slopes $\frac{1}{2}$: 1 of trapezoidal section. **Ans.**

Example 4.7. *Design an irrigation channel to carry 40 cumecs of discharge, with B/D, i.e. base width to depth ratio as 2.5. The critical velocity ratio is 1.0. Assume a suitable value of Kutter's rugosity coefficient and use Kennedy's method.*

Solution. $V_0 = 0.55\,(y)^{0.64}$ ($\because$ $m = 1$)

Here $y = D$

$\therefore$ $V_0 = 0.55 \cdot D^{0.64}$

$Q = AV$

Using $\frac{1}{2}$: 1 slopes, area (A) of trapezoidal section is given as :

$$A = BD + 2 \cdot \frac{1}{2} \cdot D \frac{D}{2} = D\left[B + \frac{D}{2}\right]$$

$$\therefore \quad 40 = D\left[B + \frac{D}{2}\right] V_0$$

But $B/D = 2.5$; or $B = 2.5D$

$$\therefore \quad 40 = D\,[2.5\,D + 0.5D]\,V_0 = D\,[3D]\,V_0 = 3D^2 \cdot V_0$$

But $V_0 = 0.55 \cdot D^{0.64}$ $\quad \therefore \quad 40 = 3D^2\,(0.55 \cdot D^{0.64})$

or $$D^{2.64} = \frac{40}{3 \times 0.55} = 24.2$$

or $$D = (24.2)^{\frac{1}{2.64}} = (24.2)^{0.379} = \mathbf{3.34\ m}$$

Now $B = 2.5\,D = 2.5 \times 3.34 = \mathbf{8.35\ m}$

Now determine the slope S

$$A = 3D^2 = 3 \times (3.34)^2 = 33.5\ \text{m}^2.$$

$$P = \left[B + 2 \cdot \frac{\sqrt{5}}{2} D\right] = (8.35 + \sqrt{5} \times 3.34) = (8.35 + 7.46) = 15.81\ \text{m}$$

$$R = \frac{33.5}{15.81} = 2.12, \text{ or } \sqrt{R} = 1.456$$

$$V_0 = 0.55\,(3.34)^{0.64} = 0.55 \times 2.163 = \mathbf{1.19}$$

Assume $n = 0.023$.

Using Eq. (4.20), we get

$$V = \left[\frac{\frac{1}{0.023} + \left(23 + \frac{0.00155}{S}\right)}{1 + \left(23 + \frac{0.00155}{S}\right)\frac{0.023}{1.456}}\right] 1.456\sqrt{S} \qquad \ldots(i)$$

Assume $S = \frac{1}{4000}$.

Putting this value of S and computing the value of V, we get

$$V = \frac{43.5 + (23 + 6.2)}{1 + 29.2 \times \frac{0.023}{1.456}} \times \frac{1.456}{63.3} = 1.114$$

$1.114 < 1.19$ or $V < V_0$

Therefore, to increase the value of V, we must increase/steepen the slope ; hence, use a slope = 1 in 3700 (say)

Putting $S = \frac{1}{3700}$ in (i) above, we get

$$V = 1.189 \approx 1.19$$

or $V = V_0$ for value of $S = \frac{1}{3700}$ $\qquad$ So use $S = \frac{1}{3700}$

Hence, use a trapezoidal channel section as follows :

$$\left.\begin{array}{r}\text{Depth} = 3.34\text{ m}\\ \text{Base width} = 8.35\text{ m}\\ \text{Side slopes} = \frac{1}{2}H : 1V\\ \text{Bed slope} = 1\text{ in }3700\end{array}\right\}$$ **Ans.**

4.7.3.1. Use of Garret's Diagrams for Applying Kennedy's Theory. A lot of mathematical calculations are required in designing irrigation channels by the use of Kennedy's method. To save mathematical calculations, graphical solution of Kennedy's and Kutter's equations, was evolved by Garret. The original diagrams given by him were in F.P.S. system, but here they have been changed into M.K.S./S.I. system. The diagrams are shown in Plates 4.1 (*a*), (*b*) and (*c*). The procedure adopted for design of irrigation channels using Garret's diagrams is explained below :

(*i*) The discharge, bed slope, rugosity coefficient, value of C.V.R. are given for the channel to be designed.

(*ii*) Find out the point of intersection of the given slope line and discharge curve. At this point of intersection, draw a vertical line intersecting the various bed width curves.

(*iii*) For different bed widths (B), the corresponding values of water depth (y) and critical velocity (V_0) can be read on the right hand ordinate. Each such pair of bed width (B) and depth (y) will satisfy Kutter's equation, and is capable of carrying the required discharge at the given slope and rugosity coefficient. Choose one such pair and determine the actual velocity of flow (V).

(*iv*) Determine the critical velocity ratio (V/V_0) taking V as calculated and V_0 as read.

(*v*) If the value of C.V.R. is not the same as given in question, repeat the procedure with other pairs of B and y.

The diagrams have been drawn for a trapezoidal channel with side slopes as $\frac{1}{2}H : 1V$ ($\frac{1}{2} : 1$) on the assumption that irrigation channels adopt approximately this shape, even though they were constructed on different side slopes.

Another important point which should be noted in these diagrams is that from the *Nomogram* provided at the top, the same curves can be used for different values of rugosity coefficient n. In the nomogram, a vertical arrow has been shown. It represents the value of n for which the curves have been drawn. When the same curves are used for some other value of n (marked on right and left sides of central value), the point of intersection of discharge and slope curves, has to be shifted to the extent given in the nomogram and also in the same direction, for drawing the vertical line.

Example 4.8. *Design an irrigation channel to carry 30 cumec of discharge. The channel is to be laid at a slope of 1 in 5000. The critical velocity ratio for the soil is 1.1. Use Kutter's rugosity coefficient as 0.0225.*

Solution. $Q = 30$ cumec, $\quad S = \dfrac{1}{5000} = 0.2 \times 10^{-3} = 0.2$ m/km

$m = 1.1, \quad n = 0.0225$

$$\frac{V}{V_0} = m = 1.1,$$

$$\therefore \quad V = 1.1\, V_0$$

Using Plate 4.1 (*c*), find out the point of intersection of the given slope line and given discharge curve. Draw a vertical line through this point. Choose a point of say bed width = 12 m, as first approximation. Calculate m as shown in the last column of Table 4.5.

Table 4.5

S. No.	*B metres*	*y metres*	V_0 *m/sec*	$A = (2B + y)\frac{y}{2} m^2$	$V = \frac{Q}{A}$ *m/sec.*	$\frac{V}{V_0} = m$
1	12.0	2.3	0.95	$(24 + 2.3)\frac{2.3}{2}$ $= 30.25$	0.99	1.04
2	12.5	2.25	0.92	30.66	0.98	1.07
3	13.0	2.15	0.90	30.26	0.99	1.1

The first approximation of using $B = 12.0$ m gives a value of m equal to 1.04 ; while as per given data its value should be 1.1. So we have to decrease V_0 in order to increase m. Therefore, in order to decrease the critical velocity, we have to reduce depth and thus to increase B. Hence, in the 2nd approximation, we increase B from 12.0 to 12.5 m, and find that m comes out to be 1.07. Again, we increase B to 13.0 m as third approximation, when we finally get the value of m equal to 1.1 (as given). Hence, choose the final values of $B = 13.0$ m, $y = 2.15$ m for the channel of bed slope 1 in 5000 and side slopes $\frac{1}{2}H : 1V$ **Ans.**

4.7.4. Lacey's Theory (1939). Lacey, an eminent civil engineer of U.P. Irrigation Department, carried out extensive investigations on the design of stable channels in alluviums. On the basis of his research work, he found many drawbacks in Kennedy's Theory (1895) and he put forward his new theory. The essential points which he argued, and the design procedure which he suggested, is briefly described here.

4.7.4.1. Lacey's regime channels. It was stated by Kennedy that a channel is said to be in a state of 'regime' if there is neither silting nor scouring in the channel. But Lacey came out with the statement that even a channel showing no silting no scouring may actually not be in regime. He, therefore, differentiated between three regime conditions : (*i*) True regime ; (*ii*) *Initial regime ;* and (*iii*) *Final regime.*

According to him, a channel which is under 'initial' regime, is not a channel in regime (though outwardly it appears to be in regime, as there is no silting or scouring) and hence, regime theory is not applicable to such channels. His theory is therefore applicable only to those channels, which are either in *true regime* or *in final regime.*

4.7.4.2. True regime. A channel shall be in regime, if there is neither silting nor scouring. For this condition to be satisfied, the silt load entering the channel must be carried through, by the channel section. Moreover, there can be only one channel section and one bed slope at which a channel carrying a given discharge and a particular quantum and type of silt, would be in regime. Hence, an artificially constructed channel having a certain fixed section and a certain fixed slope can behave in regime only if the following conditions are satisfied :

(*i*) *Discharge is constant ;*

(*ii*) *Flow is uniform ;*

(*iii*) *Silt charge is constant* ; *i.e.* the amount of silt is constant ;

(*iv*) *Silt grade is constant* ; *i.e.*, the type and size of silt is always the same, and

(*v*) *Channel is flowing through a material which can be scoured as easily as it can be deposited (such soil is known as* **incoherent alluvium***), and is of the same grade as is transported.*

Hence, a designed channel shall be in 'true regime' if the above conditions are satisfied. But in practice, all these conditions can never be satisfied. *And, therefore, artificial channels can never be in 'true regime'; they can either be in initial regime or final regime*, as explained below :

4.7.4.3. Initial regime and Final regime. When only the bed slope of a channel varies due to dropping of silt, and its cross-section or wetted perimeter remains unaffected, even then the channel can exhibit 'no silting no scouring' properties, called *Initial regime*. Thus, when water flows through an excavated channel with somewhat narrower dimensions and defective slopes, the silt carried by the water may get dropped in the upper reaches, thereby increasing the channel bed slope. Consequently, the velocity is increased, and a non-silting equilibrium is established, called 'Initial regime'. Sides of such channels are subjected to a lateral restraint and could have scoured if the bank soil would have been a true alluvium. But in practice, they may either be grassed or be of clayey soil, and therefore, they may not get eroded at all. Hence, such channels will exhibit 'non-silting, non-scouring' properties, and they will appear to be in regime ; but in fact, they are not. *They have achieved only a working stability due to the rigidity of their banks.* Their slopes and velocities are higher and cross-sections narrower than what would have been if the sides were not rigid. *Such channels are termed as channels in initial regime, and regime theory is not applicable to them, as they are infact, not the channels in alluvium.*

But, if there is no resistance from the sides, and all the variables such as perimeter, depth, slope, etc. are equally free to vary and finally get adjusted according to discharge and silt grade, then the channel is said to have achieved permanent stability, called *Final Regime*. Regime theory is applicable to such channels only, and not to all regime channels, as was envisaged by Kennedy.

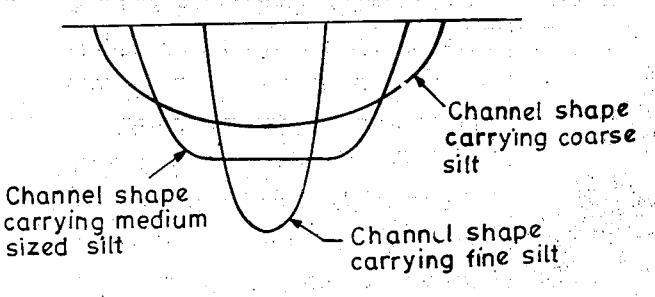


Fig. 4.10

Such a channel in which all variables are equally free to vary, has a tendency to assume a semi-elliptical section. The coarser the silt, the flatter is the semi-ellipse, *i.e.* greater is the width of the water-surface. The finer the silt, the more nearly the section attains a semi-circle (Fig. 4.10).

The second point which Lacey argued was that the sediment is kept in suspension not only by the vertical component of the eddies which are generated on the channel

---

* **incoherent** soil, as defined in article 4.4, is the soil, which does not possess any cohesion between its soil grains. The alluvial soil is defined in article 3.2, as the soil formed by continuous deposition of silt by the moving water. Such sandy soil deposits which donot possess any cohesion (c = 0) and are formed by the agency of water are called incoherent alluvium.

bed, but also by the eddies generated on the sides of the channel. Kennedy had neglected the eddies that are generated on the sides of the channel, by presuming that such eddies has horizontal movement for greater part, and, therefore, do not have sediment supporting power. Lacey thus, argued that the silt supporting power of a channel is proportional to the wetted perimeter of the channel and not to its width, as was presumed by Kennedy.

Thirdly, Lacey argued that the grain size of the material forming the channel is an important factor, and should need much more rational attention than what was given to it by Kennedy (different values of critical velocity ratio(m) for different types of soils). He, therefore, introduced a term called **silt factor** (f) in his equations, and connected it to the average particle size (as per equation (4.24)).

The various equations put forward by Lacey for the design of stable channels are given below :

4.7.4.4. Design procedure for Lacey's theory

(1) Calculate the velocity from equation

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} \text{ m/sec.} \qquad ...(4.23)$$

where Q is in cumec :
V is in m/s ; and
f is the **silt factor**, given by

$$f = 1.76 \cdot \sqrt{d_{mm}} \qquad ...(4.24)$$

where d_{mm} = Average particle size in mm, as given in Table 4.6.

Table 4.6. Values of Particle size (d_{mm}) for Various types of Alluvial materials for use in eq. (4.24)

S. No. (1)	*Type of material (soil)* (2)	*Av. grain size in mm* (d_{mm}) (3)
1	*Silt,*	
	Very fine	0.05 to 0.08
	Fine	0.12
	Medium	0.16
	Standard	0.32 (f = 1.0)
2	*Sand,*	
	Medium	0.51
	Coarse	0.73
3	*Bajri and Sand,*	
	Fine	0.89
	Medium	1.29
	Coarse	2.42
4	*Gravel,*	
	medium	7.28
	heavy	26.10
5	*Boulders,*	
	small	50.10
	medium	72.50
	large	188.80

(2) Work out the hydraulic mean depth (R) from the equation

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) \qquad ...(4.25)$$

where V is in m/sec ;
R is in m.

(3) Compute area of channel section $A = \frac{Q}{V}$ ...(4.26)

(4) Compute wetted perimeter, $P = 4.75\sqrt{Q}$ ...(4.27)

where P is in m ; Q is in m^3/sec.

(5) Knowing these values, the channel section is known ; and finally the bed slope S is determined by the equation

$$S = \left[\frac{f^{5/3}}{3340\, Q^{1/6}}\right] \qquad ...(4.28)$$

where f is the silt factor, given by Eq. (4.24)
Q is the discharge in cumec.

Lacey's Regime Width and Scour Depth for Alluvial Rivers :

For wide streams or rivers, as we know, wetted perimeter P, approximately equals the river width. Therefore, according to Lacey, for alluvial rivers :

The regime width $= W = 4.75\sqrt{Q}$ ...(4.29)

For such streams, Lacey has also defined the regime scour depth, as

$$\textit{Lacey's Normal Regime Scour Depth}^* = \mathbf{R}'_r = 0.473\left(\frac{Q}{f}\right)^{1/3} \qquad ...(4.30)$$

The above scour depth equation will be applicable only when the river width equals the regime width of $4.75\sqrt{Q}$. For any other value of active river width, the normal scour depth is given by the equation :

Lacey's Normal Scour Depth* ($\mathbf{R}'$)

$$= \mathbf{R}' = 1.35\left(\frac{q^2}{f}\right)^{1/3} \qquad ...(4.31)$$

where q is the discharge intensity per unit width of stream $= Q/L$, where L is the actual river width at the given site.

Example 4.9. *Design a regime channel for a discharge of 50 cumecs and silt factor 1.1, using Lacey's Theory.*

Solution. $Q = 50$ cumecs, $f = 1.1$

$$V = \left[\frac{Qf^2}{140}\right]^{1/6} = \left[\frac{50 \times (1.1)^2}{140}\right]^{1/6}$$

$$A = \frac{Q}{V} = \frac{50}{0.869} = 56.3\ m^2$$

* Symbol R is frequently used for representing Lacey's scour depth, but to avoid confusion with the symbol R used for the hydraulic Radius (i.e., hydraulic mean depth = A/P), we are using the symbol $\mathbf{R}'$ here for scour depth. When river width equals the regime width, then the regime scour depth is being represented by $\mathbf{R}'_r$

$$R = \frac{5}{2} \cdot \frac{V^2}{f} = \frac{5}{2} \cdot \frac{(0.869)^2}{1.1} = 1.675 \text{ m.}$$

$$P = 4.75\sqrt{Q} = 4.75 \cdot \sqrt{50} = 33.56 \text{ m}$$

For a trapezoidal channel with $\frac{1}{2}H : 1V$ slopes

$$P = b + \sqrt{5} \cdot y$$

and $$A = \left(b + \frac{y}{2}\right) y$$

$$\therefore \quad 33.56 = b + \sqrt{5} \cdot y \qquad \ldots(i)$$

and $$56.3 = by + \frac{y^2}{2} \qquad \ldots(ii)$$

From Eq. (*i*), we get, $b = 33.56 - 2.24y$

Putting this value of b in Eq. (*ii*)

$$56.3 = [33.56 - 2.24y]\, y + \frac{y^2}{2}$$

$$= 33.56y - 2.24y^2 + 0.5y^2 = 33.56y - 1.74y^2$$

or $$1.74y^2 - 33.56y + 56.3 = 0$$

or $$y^2 - 19.3y + 32.4 = 0$$

$$\therefore \quad y = \frac{19.3 \pm \sqrt{372 - 129.6}}{2}$$

$$= \frac{19.3 \pm \sqrt{242.4}}{2} = \frac{19.3 \pm 15.6}{2}$$

Neglecting unfeasible + ve sign, we get

$$y = \frac{19.3 - 15.6}{2} = 1.65 \text{ m}$$

$$\therefore \quad y = 1.65 \text{ m.} \quad \textbf{Ans.}$$

$$b = 33.56 - 2.24 \times 1.65 = \mathbf{29.77\ m}$$

or $$b = 29.77 \text{ m.} \quad \textbf{Ans.}$$

$$S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{(1.1)^{5/3}}{3340 \cdot (50)^{1/6}} = \frac{1}{5420}$$

Use a bed slope of 1 in 5420. **Ans.**

4.7.4.5. Use of Lacey's diagrams. Lacey's equations for design of irrigation channels (explained earlier) have been converted into graphical solutions. Pl see enclosed Plate Figs 4.2 (*a*), (*b*) and (*c*) pasted on pages 118-119. From these diagrams, the slopes and dimensions of any channel can be easily determined, if discharge and silt factor are known.

4.7.5. Comparison of Kennedy's and Lacey's Theories and Improvements over Lacey's Theory. (1) The concept of silt transportation is the same in both the cases. Both the theories agree that the silt is carried by the vertical component of the eddies generated by the friction of the flowing water against the channel surface. The difference is that Kennedy considered a trapezoidal channel section and, therefore, he neglected the eddies generated from the sides, on the presumption that these eddies has horizontal movement for greater part and, therefore, did not have silt supporting power. For this

reason, kennedy's critical velocity formula was derived only in terms of depth of flow (y). On the other hand, Lacey considered that an irrigation channel achieves a cup-shaped section (semi-ellipse) and that the entire wetted perimeter (P) of the channel contributes to the generation of silt supporting eddies. He, therefore, used hydraulic mean radius $\left(R = \frac{A}{P} \right)$ as a variable in his regime velocity formula instead of depth (y).

(2) Kennedy stated all the channels to be in a state of regime provided they did not silt or scour. But, Lacey differentiated between the two regime conditions, *i.e.* Initial regime and final regime.

(3) According to Lacey, the grain size of the material forming the channel is an important factor and should need much more rational attention than what was given to it by Kennedy. Kennedy has simply stated that Critical velocity ratio ($V/V_0 = m$) varies according to the silt conditions (*i.e.* silt grade and silt charge). Lacey, however, connected the grain size (d) with his silt factor (f) by the equation $f = 1.76 \cdot \sqrt{d_{mm}}$. The silt factor ($f$) occurs in all those Lacey's equations, which are used to determine channel dimensions.

(4) Kennedy has used Kutter's formula for determining the actual generated channel velocity. The value of Kutter's rugosity coefficient (n) is again a guess work. Lacey, on the other hand, after analysing huge data on regime channels, has produced a general regime flow equation, stating that

$$V = 10.8\, R^{2/3}\, S^{1/3} \qquad \text{...(4.32)}$$

(5) Kennedy has not given any importance to bed width and depth ratio. Lacey has connected wetted perimeter (P) as well as area (A) of the channel with discharge, thus, establishing a fixed relationship between bed width and depth.

(6) Kennedy did not fix regime slopes for his channels, although his diagrams indicate that *steeper slopes* are required for *smaller channels* and flatter slopes are required for larger channels. Lacey, on the other hand, has fixed the regime slope, connecting it with discharge by the formula given by eqn. (4.28) as

$$S = \frac{f^{5/3}}{3340 \cdot Q^{1/6}}, \qquad \text{(Eq. 4.28)}$$

This regime slope formula, given by Lacey, gives excessive slope values. Not even a single channel has been constructed according to this regime slope equation, either on the lower Chenab Canal System or on the Lower Bari Doab Canal System or on the Jhelum Canal System. The rigidity of this regime slope equation was, therefore, later changed by Lacey (in C.B.I. publication No. 20) to the form

$$S \propto \frac{f^{5/3}}{q^{1/3}} \qquad \text{...(4.28 } a\text{)}$$

which was his final formula, showing no rigidity of the constants.

ESTIMATION OF TRANSPORTED SEDIMENT IN A CANAL

The quantity of sediment entering a channel from the head-works is an important factor. It fully controls the cross-section and shape of the true regime channel. Regime theories do not consider this vital important factor in channel design. *However, it has now been realised that the channel design shall not be successful unless full provision is made for the effects produced by the actual quantity of sediment moving in the channel. Hence, a detailed study of the subject becomes important.*

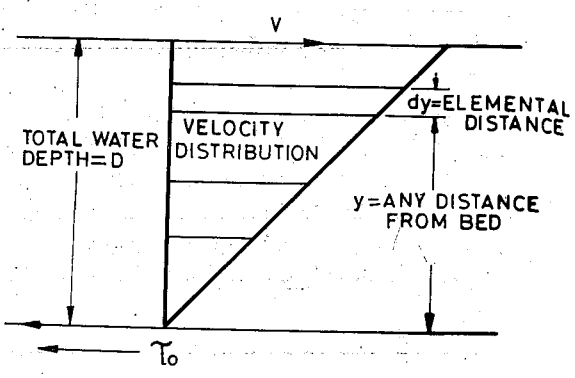
The sediment moving in water has already been classified into

(*i*) *Bed Load*, and (*ii*) *Suspended Load*

We shall now derive equations for evaluating the amounts of these loads in a canal.

4.8. Suspended Load and its Measurement

After the threshold of motion has been passed due to increase in flow velocity, and the sediment movement is well established, some of the sediment will be carried in suspension. *The material is kept in suspension by the turbulence, or in other words, by the generation of the eddies that rise from the regions of higher sediment concentration to the regions of lower sediment concentrations.*



τ_0 = Shear stress at bed = Rate of change of velocity with distance $= \mu \frac{\partial V}{\partial y}$

Fig. 4.11

In a laminar flow, the shear stress caused at the base is due to one factor, and that is due to the difference of velocities at the top and at the base, as shown in Fig. 4.11. Therefore,

$\tau_0 = \mu \frac{\partial V}{\partial y}$ for a laminar flow,

where μ is the **dynamic viscosity** of the water (fluid.) But in a turbulent flow, another factor that comes into play, is the jumping of particles from higher velocity region to lower velocity region. This is known as momentum transfer or mass exchange. Due to this transference of mass or momentum between the two adjacent fluid layers, an effective shear stress is caused at the interface between the layers, as shown in Fig. 4.12.

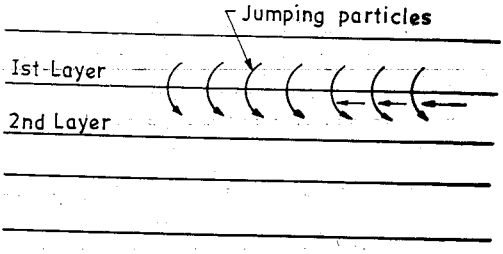
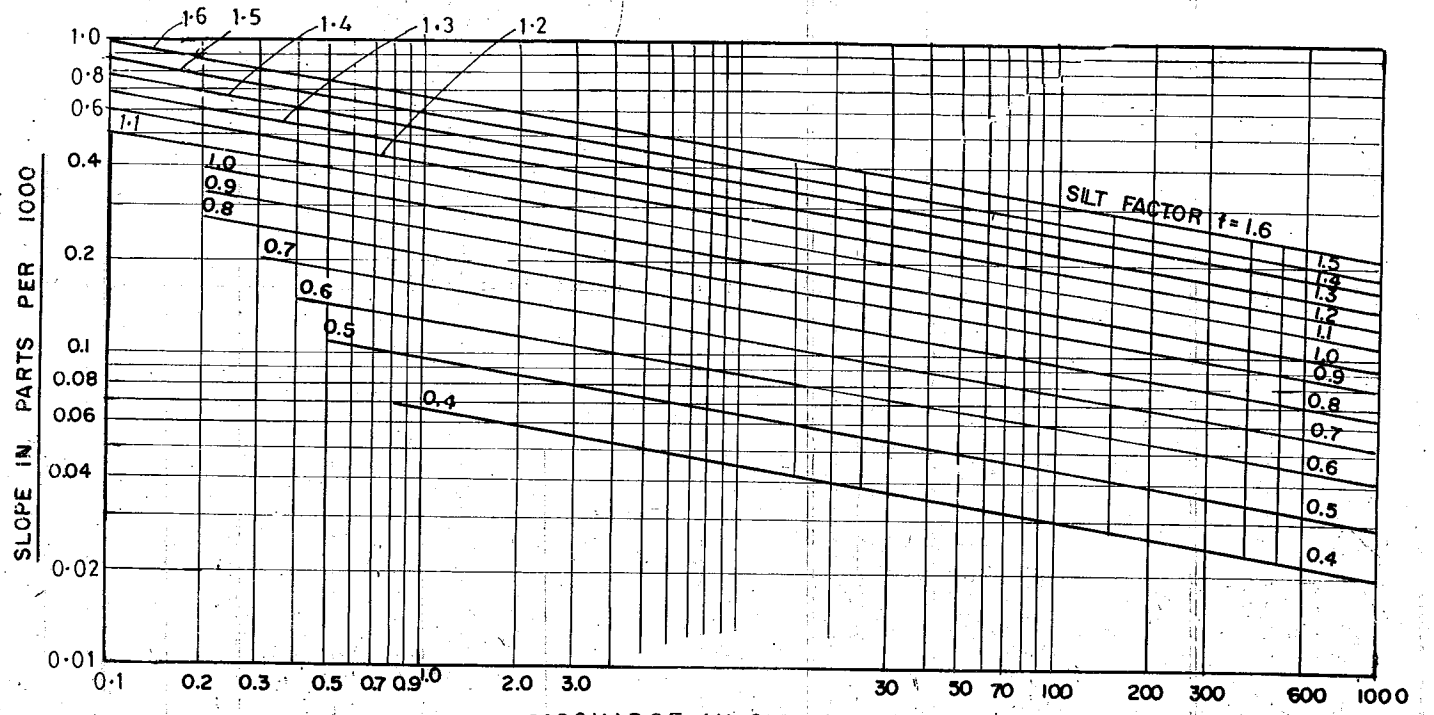


Fig. 4.12

The shear stress produced by this action is given by $\eta \cdot \frac{dV}{dy}$, where η is the **eddy viscosity,** and is defined as the rate of *mass* exchange per unit area between the adjacent layers.

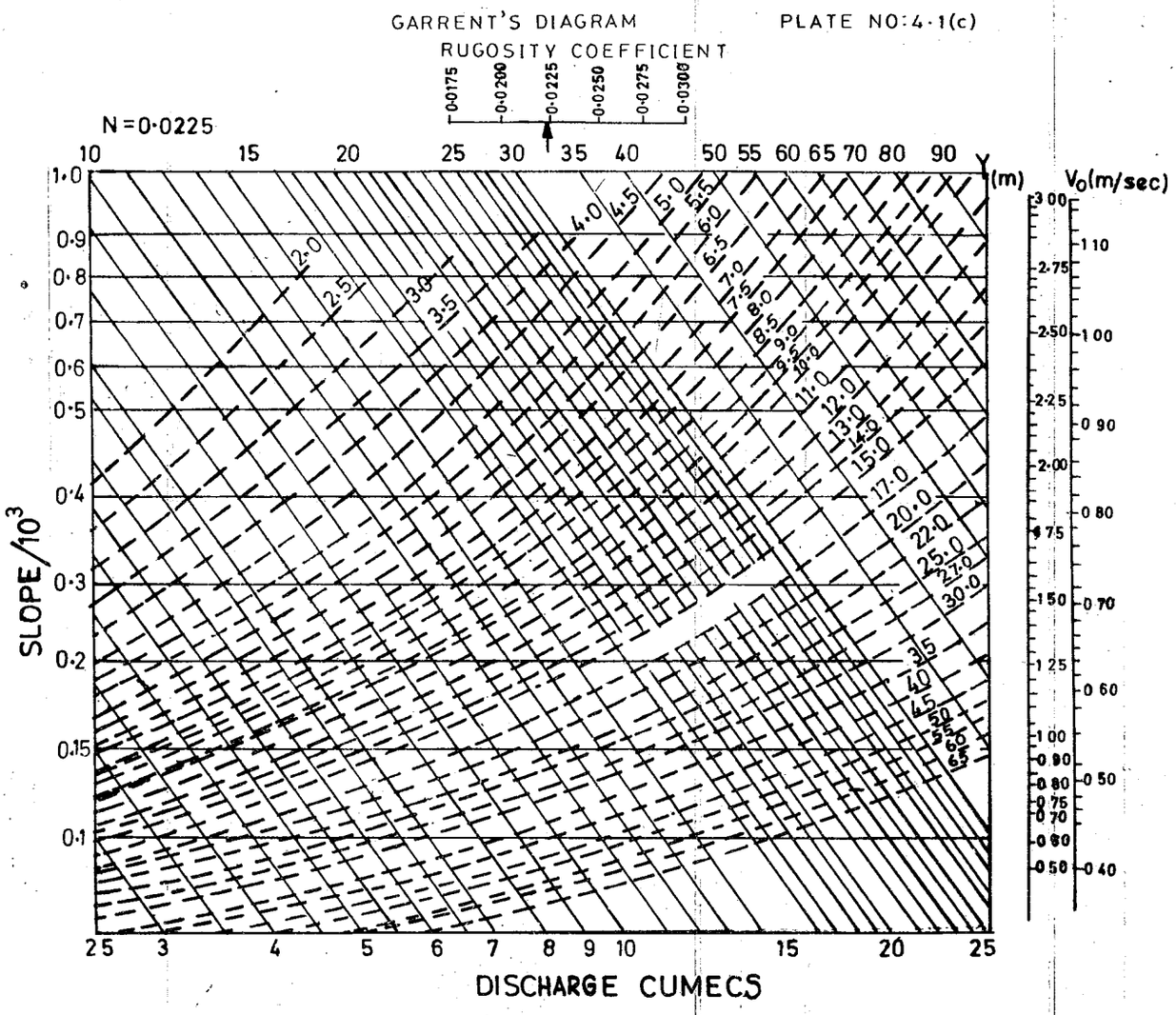
Just as we define *kinematic viscosity of water* $\nu = \left(\frac{\mu}{\rho_w}\right)$ we here define $\varepsilon = \frac{\eta}{\rho_w}$ where ε is defined as the *rate of volume exchange per unit area,* between the *adjacent layers,* and is called **eddy kinematic viscosity.**

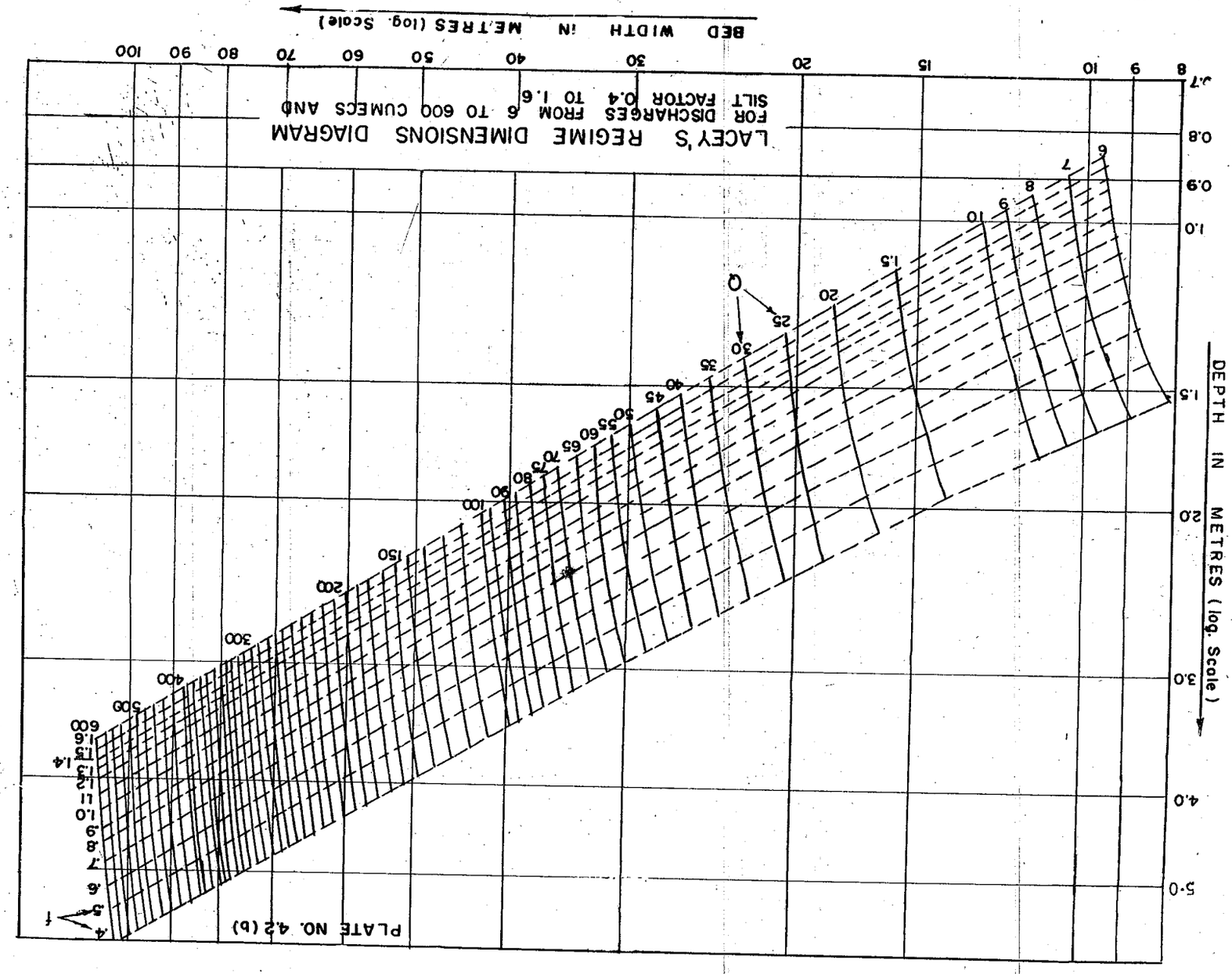
This momentum transfer caused by the generation of eddies brings about the transfer of sediment from the regions of higher concentration to the regions of lower concentration.



LACEY'S REGIME SLOPE DIAGRAM

GARRENT'S DIAGRAM PLATE NO: 4·1(c)

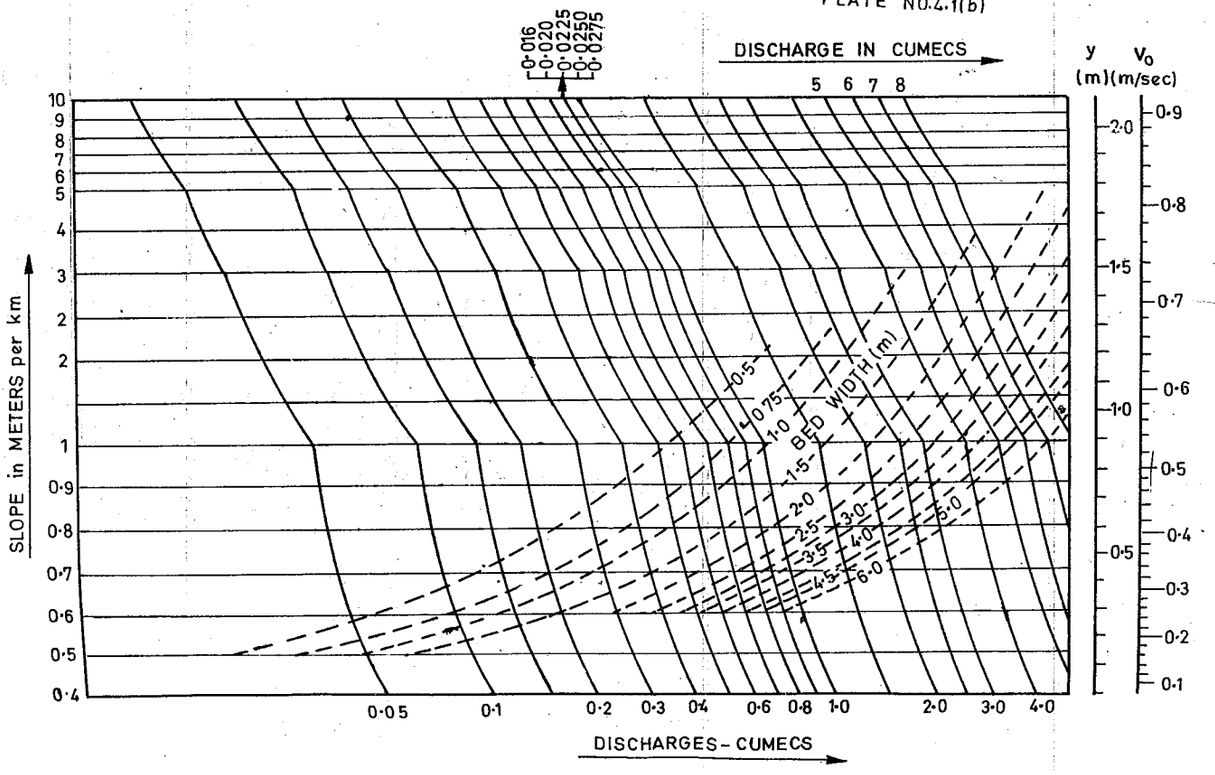




GARRET'S DIAGRAM

RUGOSITY COEFFICIENT

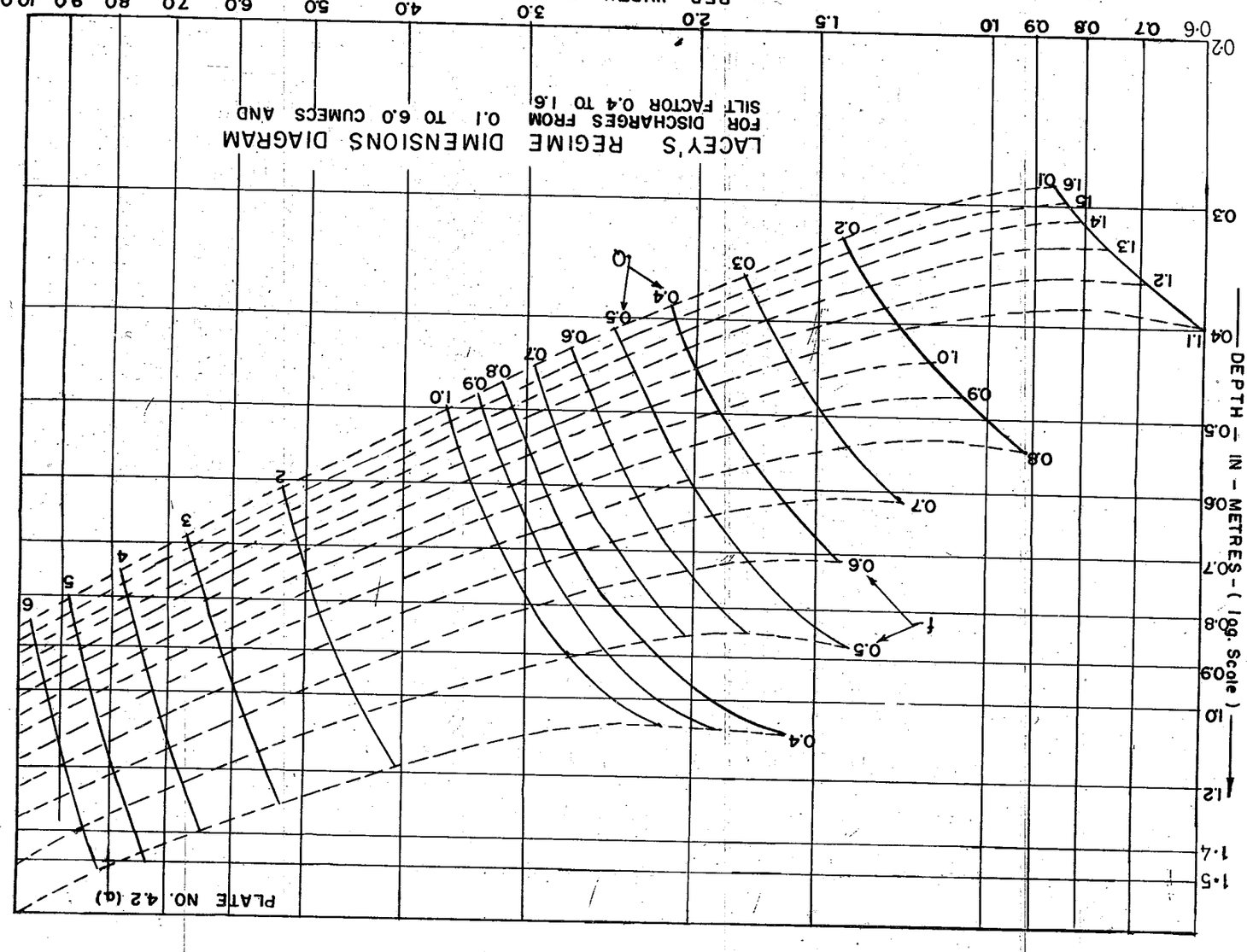
PLATE NO.4.1(b)

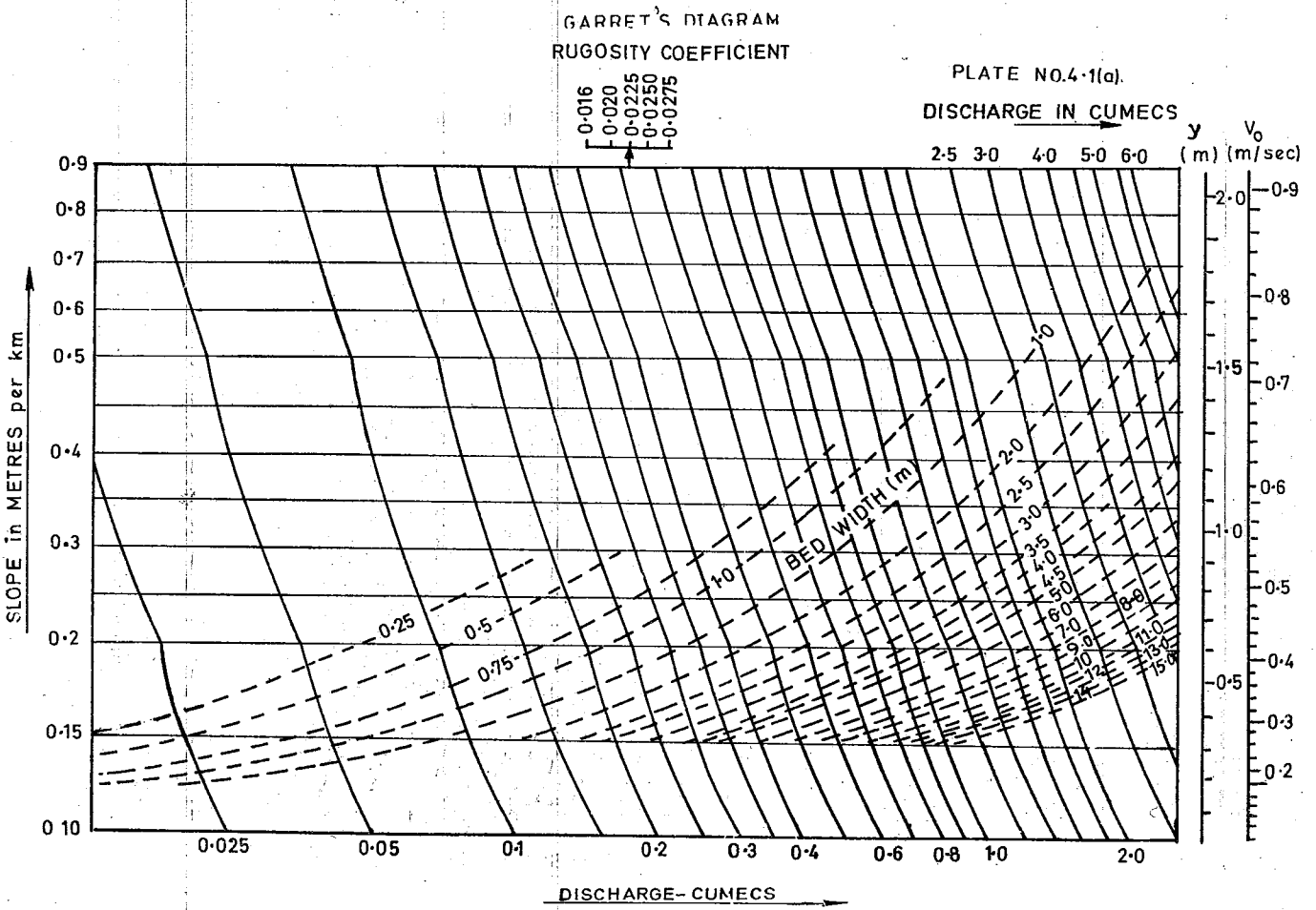


LACEY'S REGIME DIMENSIONS DIAGRAM

FOR DISCHARGES FROM 0.1 TO 6.0 CUMECS AND

SILT FACTOR 0.4 TO 1.6




GARRET'S DIAGRAM
RUGOSITY COEFFICIENT
PLATE NO.4·1(a).
0·016
0·020
0·0225
0·0250
0·0275
DISCHARGE IN CUMECS
y (m)
V0 (m/sec)
2·5 3·0 4·0 5·0 6·0
SLOPE in METRES per km
0·9
0·8
0·7
0·6
0·5
0·4
0·3
0·2
0·15
0 10
BED WIDTH (m)
0·25
0·5
0·75
1·0
2·0
2·5
3·0
3·5
4·0
4·5
5·0
6·0
7·0
8·0
9·0
10
11·0
12
13·0
14
15·0
0·025
0·05
0·1
0·2
0·3
0·4
0·6
0·8
1·0
2·0
DISCHARGE-CUMECS

If $\frac{dc}{dy}$ represents the gradient of sediment concentration between the adjacent layers (Fig. 4.13), then the rate of sediment transfer (in terms of mass between the adjacent layers)

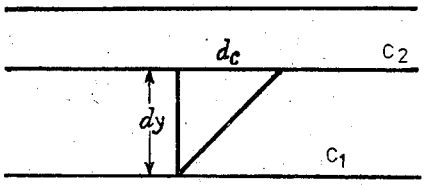


Fig. 4.13.

$$= -\eta \frac{dc}{dy}.$$

(– ve sign indicates that concentration is decreasing in the positive direction of y.)

$\therefore$ Rate of sediment transfer (by volume between the adjacent layers)

$$= -\frac{\eta}{\rho_w} \cdot \frac{dc}{dy} = -\varepsilon \cdot \frac{dc}{dy}$$

The sediment moved up by these eddies is brought down by its self-weight.

The rate of sediment falling down due to gravity

$= w_0 c$ where w_0 is the velocity of free fall of the grain, and c is sediment concentration.

When the material is just under suspension, the two forces must be equal.

Therefore, $-\varepsilon \frac{dc}{dy} = w_0 c$

or $$w_0 c + \varepsilon \frac{dc}{dy} = 0 \qquad ...(4.33)$$

is the governing differential equation.

Integrating both sides after separating the variables, we get

$$\int \frac{dc}{c} = -\int \frac{w_0}{\varepsilon} dy$$

Two cases now arise.

Case I. *When* ε *(eddy kinematic viscosity) is constant*

$$\int \frac{dc}{c} = -\frac{w_0}{\varepsilon} \int dy$$

or $$\log_e c = -\frac{w_0}{\varepsilon} \cdot y + k \qquad ...(i)$$

If c_a is the sediment concentration at a distance a from the bed, then

$$\log_e c_a = -\frac{w_0}{\varepsilon} \cdot (a) + k$$

or $$k = \log_e c_a + \frac{w_0}{\varepsilon} (a)$$

using this value of k; the equation (i) reduces to

$$\log_e c = -\frac{w_0}{\varepsilon} \cdot y + \left(\log_e c_a + \frac{w_0}{\varepsilon} \cdot a \right)$$

or $$\log_e c - \log_e c_a = -\frac{w_0}{\varepsilon} \cdot y + \frac{w_0}{\varepsilon} \cdot a$$

or $$\log_e \frac{c}{c_a} = -\frac{w_0}{\varepsilon} \cdot (y - a)$$

or $$\boxed{\frac{c}{c_a} = e^{-\frac{w_0}{\varepsilon}(y-a)}} \qquad ...(4.34)$$

Hence, the sediment concentration c at any distance y can be determined, if the concentration c_a at a given distance a is known.

Case II. *When ε (eddy kinematic viscosity) is not constant.*

Generally, ε is not constant, and it has been shown by Prandtl that $\varepsilon = l^2 \cdot \frac{dV}{dy}$ where, l is **'mixing length'** and is analogous to 'mean free path' of kinetic theory of gases. The mixing length is a measure of turbulence and is an indicator of the size of the generated eddies.

Now, we know that $c = (\mu + \eta) \cdot \frac{dV}{dy}$.

But μ is very small compared to η, and can be neglected.

Therefore, $$c = \eta \frac{dV}{dy}$$

In the close neighbourhood of a solid boundary, it has been found from dimensional considerations that the 'Mixing length' (l) is directly proportional to the distance (y) from the boundary

or $l \propto y$ or $l = Ky$

where K is known as **Von-Karman Universal constant** and is equal to 0.4 for all homogeneous fluids.

Now, $$\tau = \eta \cdot \frac{dV}{dy} = \rho_w \cdot \varepsilon \cdot \frac{dV}{dy} = \rho_w \cdot \left(l^2 \cdot \frac{dV}{dy}\right)\frac{dV}{dy}$$

$$= \rho_w \cdot (Ky)^2 \cdot \left(\frac{dV}{dy}\right)^2 = \rho_w K^2 \cdot y^2 \cdot \left(\frac{dV}{dy}\right)^2.$$

Again in the same immediate neighbourhood, we can assume,

$$\tau = \tau_0.$$

$\therefore$ $$\tau_0 = \rho_w \cdot K^2 \cdot y^2 \cdot \left(\frac{dV}{dy}\right)^2 \quad \text{or} \quad \sqrt{\frac{\tau_0}{\rho_w}} = K \cdot y \cdot \frac{dV}{dy}$$

But $\sqrt{\frac{\tau_0}{\rho_w}} = V^*$ $\therefore$ $V^* = K \cdot y \cdot \frac{dV}{dy}$

or $$\boxed{\frac{dV}{dy} = \frac{V^*}{K \cdot y}} \qquad ...(4.35)$$

The equation (4.35) gives the well known law of logarithmic velocity distribution ; given by

$$V = \frac{V^*}{K} \log_e y + \text{constant} \qquad ...(4.36)$$

But, in fact, the shear stress at any distance y from the boundary is given by

$$\tau = \tau_0 \left(1 - \frac{y}{D}\right) \qquad ...(4.37)$$

where D is the total depth.

Also assume that at sufficient distance (y) from the boundary, the mixing length is given by

$$l = K \cdot y \cdot \sqrt{1 - \frac{y}{D}}$$

Now, $$\varepsilon = l^2 \cdot \frac{dV}{dy} = K^2 \cdot y^2 \cdot \left(1 - \frac{y}{D}\right) \cdot \frac{dV}{dy}$$

But $$\frac{dV}{dy} = \frac{V^*}{K \cdot y} \text{ from equation (4.35)}$$

$\therefore$ $$\varepsilon = K^2 \cdot y^2 \cdot \left[1 - \frac{y}{D}\right] \cdot \frac{V^*}{Ky}$$

or $$\varepsilon = V^* Ky \left[1 - \frac{y}{D}\right] \qquad ...(4.38)$$

The differential equation (4.33) was

$$w_0 c + \varepsilon \cdot \frac{dc}{dy} = 0$$

Putting the value of ε from Eqn. (4.38) in the above equation, we get

$$w_0 c + V^* K \cdot y \left[1 - \frac{y}{D}\right] \cdot \frac{dc}{dy} = 0 \quad \text{or} \quad w_0 c + V^* K \cdot y \left[\frac{D - y}{D}\right] \cdot \frac{dc}{dy} = 0$$

or $$D \cdot w_0 c + V^* Ky (D - y) \cdot \frac{dc}{dy} = 0 \quad \text{or} \quad D \cdot w_0 c = - V^* Ky (D - y) \cdot \frac{dc}{dy}$$

or $$\frac{D \cdot dy}{y (D - y)} = - \frac{V^* K}{w_0} \cdot \frac{dc}{c}$$

Integrating, we get

$$D \cdot \int \frac{dy}{y (D - y)} = - \frac{V^* \cdot K}{w_0} \int \frac{dc}{c} \qquad ...(4.39)$$

Let us evaluate the L.H.S. integral first, *i.e.*

$$D \cdot \int \frac{dy}{y (D - y)}.$$

Let $$\frac{1}{y (D - y)} = \frac{A}{y} + \frac{B}{D - y}$$

or $$A (D - y) + B \cdot y = 1$$

Equating coefficients on both sides, we get

$$AD = 1, \qquad \text{or} \qquad A = \frac{1}{D}$$

and $$-A + B = 0 \qquad \text{or} \qquad B = A = \frac{1}{D}$$

$$\therefore \quad \frac{1}{y\,(D-y)} = \frac{1}{D \cdot y} + \frac{1}{D\,(D-y)} = \frac{1}{D}\left[\frac{1}{y} + \frac{1}{D-y}\right]$$

or $$D\int \frac{dy}{y\,(D-y)} = D \cdot \frac{1}{D} \cdot \int \left[\frac{1}{y} + \frac{1}{D-y}\right] dy = \int \frac{1}{y}\, dy + \int \frac{1}{D-y}\, dy$$

$$= \log_e y - \log_e \overline{D-y} = \log_e \left(\frac{y}{D-y}\right)$$

Putting this value in Eqn. (4.39), we get

$$\log_e \left(\frac{y}{D-y}\right) = -\frac{V^* K}{w_0} \int \frac{dc}{c}$$

or $$\log_e \left(\frac{y}{D-y}\right) = -\frac{V^* K}{w_o} \log_e c + \text{constant of integration}$$

or $$\log_e \left(\frac{y}{D-y}\right) = -\frac{V^* K}{w_0} \log_e c + K'$$

or $$-\log_e c = \frac{w_0}{V^* K} \log_e \left(\frac{y}{D-y}\right) + K'' \qquad \ldots(i)$$

If $y = a$, $c = c_a$

$$\therefore \quad -\log_e c_a = \frac{w_0}{KV^*} \log_e \left(\frac{a}{D-a}\right) + K''$$

or $$K'' = -\log_e c_a - \frac{w_0}{KV^*} \log_e \left(\frac{a}{D-a}\right) \qquad \ldots(ii)$$

Putting this value of K'' in (i), we get

$$-\log_e c = \frac{w_0}{KV^*} \log_e \left(\frac{y}{D-y}\right) - \log_e c_a - \frac{w_0}{KV^*} \log_e \left(\frac{a}{D-a}\right)$$

or $$\log_e c - \log_e c_a = \frac{w_0}{KV^*} \log_e \left(\frac{a}{D-a}\right) - \frac{w_0}{KV^*} \log_e \frac{y}{D-y}$$

or $$\log_e \frac{c}{c_a} = \frac{w_0}{KV^*} \left[\log_e \frac{a}{D-a} \times \frac{D-y}{y}\right] = \log_e \left[\frac{a(D-y)}{y\,(D-a)}\right]^{\frac{w_0}{KV^*}}$$

or $$\boxed{\frac{c}{c_a} = \left[\frac{a\,(D-y)}{y\,(D-a)}\right]^{\frac{w_0}{KV^*}}} \qquad \ldots(4.40)$$

where, c is the sediment concentration at a distance y, and this can be determined by the above **Rouse equation,** by knowing the sediment concentration (c_a) at a known distance 'a' apart ; D being the total water depth.

w_0 is the fall velocity of a grain in still water, and V^* is the **shear friction velocity,** given by

$$V^* = \sqrt{\frac{\tau_0}{\rho_w}} \qquad ...(4.41)$$

where τ_0 is the shear stress at the bottom, and ρ_w is the density of water.

K is the Von-Karman Universal Constant = 0.4.

D is the total water depth.

The equation (4.40) is known as the *suspended load concentration equation.* The limitation of this equation is that it cannot be used directly to predict the sediment concentration at any point, unless the sediment concentration at some known distance 'a' above the channel bed is preknown. Moreover, this equation has been derived on the assumptions of : (*i*) *two dimensional steady flow;* (*ii*) Constant fall velocity (w_0), and (*iii*) Constant Value of Karman's Constant (K) ; although however, it is a known fact today, that both w_0 as well as K vary with sediment concentration and turbulence.

Inspite of its above limitations, this equation (4.40) remains an important equation for determining sediment concentration in a channel at any depth y above the bed of the channel. The computation of sediment concentrations at different depths above the bed of the channel, can further be used to evaluate the total suspended load transported by the channel, as discussed below :

Rate of suspended load transport. As stated above, equation (4.40) will help us to compute the sediment concentrations at different depths, and thus plot a curve between sediment concentration (c) and depth (y) as shown in Fig. 4.14 (*a*). This curve is known as **sediment concentration curve.** When once this variation of sediment

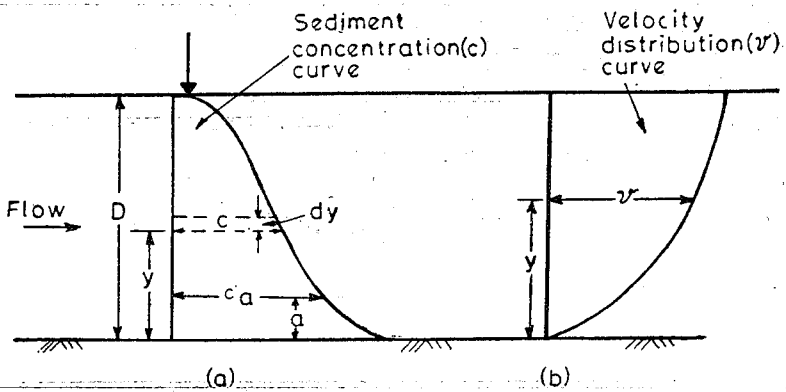


Fig. 4.14. Sediment concentration and Velocity distribution curves for a channel.

concentration with depth is known, it can be integrated over the depth to obtain total suspended load. This integration also requires plotting of **velocity distribution curve,** which is shown in Fig. 4.14 (*b*).

With reference to Fig. 4.14, let us consider a flow strip of unit width and of thickness dy at an elevation y.

The volume of suspended sediment transported past this strip in a unit time

$$= \left(\frac{c}{100}\right) v \cdot dy \qquad \text{...(4.41)}$$

where c = The sediment concentration (volume expressed as percentage at any elevation y, where the flow velocity is v.

or $$q_s = \int \frac{c}{100} \cdot v \cdot dy \qquad \text{...[4.42 (a)]}$$

Since the suspended sediment moves only on top of the bed layers, the lower limit of integration can be considered as equal to the thickness of the bed layer, which is approximated to be equal to $2d^*$ (d being grain size), and the upper limit will certainly be D (total water depth). Hence,

$$q_s = \int_{y=2d}^{y=D} \frac{c}{100} \cdot v \cdot dy \qquad \text{...(4.42)}$$

where q_s = Rate of suspended load transported in m^3/sec.

The reference concentration c_a required to compute various values of c by Eq. (4.40), as pointed out earlier, needs to be predetermined for the above computations. When this depth 'a' is considered equal to '$2d$', the sediment concentration c_a or 'c_{2d}' can be considered to be equal to the concentration of bed load, since bed load is assumed to be existing up to '$2d$' above the bed*. On the basis of computations carried out for bed load rate (q_b) in m^3/sec, discussed in the next article, Einstein has further computed this sediment concentration c_a at depth $a = 2d$ above bed, as equal to:

$$c_a = c_{2d} = \frac{q_b}{23.2 V^* d} \times 100 \qquad \text{...(4.43)}$$

where V^* = shear friction velocity

$$= \sqrt{\frac{\tau_0}{\gamma_w}} = \sqrt{\frac{\gamma_w \cdot R' \cdot S}{\rho_w}}$$

$$= \sqrt{\frac{\rho_w \cdot g \cdot R' S}{\rho_w}} = \sqrt{gR'S} \qquad \text{...(4.44)}$$

where R' is the modified value of R due to ripples, as explained in the next article, and equals $R' = R \cdot \left(\frac{n'}{n}\right)^{3/2}$ as per equation (4.47).

Example 4.10. *In a wide stream, a suspended load sample taken at a height of 0.30 m from the bed indicated a concentration of 1000 ppm of sediment by weight. The stream is 5.0 m deep and has a bed slope of 1/4000. The bed material can be assumed to be of*

* Sediment confined along and above the bed, up to a depth 2.d (d being grain size), is treated as bed load (q_b).

uniform size with a fall velocity of 2.0 cm/s. Estimate the concentration of the suspended load at mid-depth. (*Civil Services, 1991*)

Solution. Using Eq. (4.40), we have

$$\frac{c}{c_a} = \left[\frac{a(D-y)}{y(D-a)}\right]^{\frac{w_0}{KV^*}} \qquad \text{(Eq. 4.40)}$$

where $D = 5$ m

$$S = \frac{1}{4000}$$

y = up distance from bed of channel, where sediment concentration c is desired *i.e.* at mid depth $= \frac{5}{2} = 2.5$ m

c_a = known sediment concentration = 1000 ppm (of sediment by wt.)

a = known distance where sediment concentration is $c_a = 0.3$ m (given)

w_0 = fall velocity of sediment grain in still water = 2.0 cm/s (given) = 0.02 m/s

K = Von-karman constant = 0.4 (assumed)

V^* = shear friction velocity

$$= \sqrt{\frac{\tau_0}{\rho_w}} = \sqrt{\frac{\gamma_w RS}{\rho_w}} = \sqrt{\frac{\rho_w gRS}{\rho_w}} = \sqrt{g \cdot RS}$$

where $R = D$ for wide channels = 5 m.

or $$V^* = \sqrt{9.81 \times 5 \times \frac{1}{4000}} = 0.111 \text{ m/s}$$

$c = ?$

Using the above equation (4.40), we have

$$\frac{c}{1000 \text{ ppm}} = \left[\frac{0.3(5-2.5)}{2.5(5-0.3)}\right]^{\frac{0.02}{0.4 \times 0.111}} = \left(\frac{0.75}{11.75}\right)^{0.45} = 0.29$$

or $c = \mathbf{290 \text{ ppm.}}$ **Ans.**

4.9. Bed Load and Its Measurement

Bed load, as explained earlier, is the sediment material that remains in the bottom layers of the flow, and its movement takes place by *rolling, sliding,* and *hopping* (*i.e.* **saltation**) depending upon the velocity of flow. The bed load can be measured, though quite unsatisfactorily, by various **samplers,** such as *box type sampler, slot type sampler,* etc. Bed load is also sometimes estimated by assuming it to be between 3 to 25% of the total suspended load, depending upon the nature of response of the load material to the forces from physical, chemical and biological sectors. A figure of 10% is more commonly adopted.

Mathematical equations for bed load. The suspended load is the sediment which remains suspended in the water flowing in the channel, and this is caused by the forces

of turbulence generated by the flow. The bed load, on the other hand, is the sediment which moves along the bed of the channel, and this movement is caused by the shear stress (τ_0) developed by the flowing water along the channel bed. This shear force, called the *drag force* or the *tractive force* is thus fully responsible for the bed load movement. Evidently, the amount of bed load transported in a channel of given R and S will depend on the shear stress (τ_0) developed on the bed of that channel. This bed shear or unit tractive force (τ_0) is given by the already indicated equation (4.1) as :

$$\tau_0 = \gamma_w RS$$

where R = Hydraulic mean depth of the channel $\simeq y$ (or D) for wider channels.
S = Bed slope
γ_w = Unit wt. of water = 9.81 kN/m^3
= 9.81×10^3 N/m^3

We further know that a certain minimum value of shear stress is required to move the grain, depending upon the internal friction of soil. It is called critical shear stress and is represented by τ_c. For usual turbulent flow and for quartz grains, the value of τ_c has been approximated as

$$\tau_c = 0.687\, d_a \qquad ...(4.45)$$

where d_a = arithmatic average diameter of sediment in mm (generally varying between d_{50} and d_{60}).
τ_c = critical shear stress in N/m^2.

when the unit tractive force caused by the flowing water (τ_0) exceeds the critical unit tractive force (τ_c) ; naturally, sediment starts moving. The rate of bed-load transported, must, therefore, be a function of ($\tau_0 - \tau_c$). But the problem becomes complicated, when we take into account the fact that as soon as the grain starts moving, the channel bed develops ripples, and a large part of shearing force is absorbed by the form resistance caused by these ripples. A part of the tractive force is, therefore, lost in overcoming ripples, and it does not play any role in transporting bed material. The quantity of shear stress lost in this process is unknown, and no perfect mathematical solution has been but forward to work out this quantity.

It has been widely suggested that the tractive force is reduced by ripples, in the following ratio :

$$\tau_0' = \tau_0 \left[\frac{n'}{n}\right]^{3/2} \qquad ...(4.46)$$

where τ_0' = Resultant unit tractive force, left after the ripple resistance is overcome (*i.e.*, τ_0 – tractive force lost in overcoming ripples).
τ_c = Original tractive force exerted by the water flowing in the channel of given R and S, and is equal to $\gamma_w RS$.

n' = Rugosity coefficient that should come into play, theoretically, in an unrippled channel of given R and S. Its value may be obtained by Strickler's formula i.e. $n' = \frac{1}{24} d^{1/6}$, where d is the effective grain diameter *i.e.* **median size** (d_{50}) of the bed sediment in metres.

n = Rugosity coefficient, actually observed by experiments on the rippled bed of the channel and its value is generally taken as 0.020 for discharges over 11 cumecs, and 0.0225 for smaller canals.

From Eqn. (4.46), and using $\tau_0 = \gamma_w RS$ from Eq. (4.1), we get

$$\tau_0' = \gamma_w RS \cdot \left[\frac{n'}{n}\right]^{3/2}$$

or $$\tau_0' = \gamma_w \left[R\left(\frac{n'}{n}\right)^{3/2}\right] S$$

or $$\tau_0' = \gamma_w R' S.$$

$$\text{where } R' = R \cdot \left(\frac{n'}{n}\right)^{3/2} \qquad ...(4.47)$$

where R' is the corresponding hydraulic mean depth that would exist in the channel, if the bed was unrippled. In other words, if we use the value of R' in all our calculations instead of R, we can forget about bed ripples, *i.e.* the effect of ripples is only to reduce the hydraulic mean depth to a value R' from R.

Certain empirical formulas, used these days for determining bed load transport, are given below. The Einstein's semi-theoretical formula based on various assumptions is the most important and widely used these days.

4.9.1. Empirical Formula by DU-Bois. The first equation on the rate of bed load transport was proposed by DU-Bois, who assumed that the rate of bed load transportation was proportional to the excess of prevailing tractive force over the critical value required to initiate movement. Thus,

$$q_b = K_b \cdot \tau_0 (\tau_0 - \tau_c) \qquad ...(4.48)$$

where q_b = Bed load (volume) transported in m^3 per second per unit width of channel.

τ_0 = Average shear stress on the channel boundary (N/m^2).

τ_c = Minimum shear stress required to move the grain, called critical shear stress (N/m^2).

K_b = a constant depending upon the grain size, and given as :

$$K_b = \frac{1.798 \times 10^{-3}}{(d)^{3/4}}$$, where d is the effective grain diameter *i.e.* median size (d_{50}) of the bed sediment, in mm ...(4.48*a*)

4.9.2. Shield's Formula (1936). A dimensionally homogeneous equation for sediment of uniform size, taking into account the effect of specific gravity of sediment (S_s), was proposed by shield as :

$$\frac{qS_s}{q_b} = 10\left[\frac{\tau_0 - \tau_c}{\gamma_w \cdot d\,(S_s - 1)}\right] \quad ...(4.49)$$

where q_b = Bed load transported in m^3/sec per m width of channel

S_s = Specific gravity of the bed grain

q = Discharge per unit width in m^2/sec

γ_w = Unit weight of fluid in kN/m^3

d = Dia. of bed grain in m

τ_o and τ_c are stresses in kN/m^2.

4.9.3. Meyer-Peter's Formula (1948). On the lines already discussed, Meyer and Peter has suggested that the unit tractive force causing bed load to move, is reduced by ripples, in the ratio of

$$\tau_0' = \tau_0 \left(\frac{n'}{n}\right)^{3/2} \quad ...(4.50)$$

The effective unit tractive force going to cause bed load transportation, is then given by

$$\tau_{eff} = \left[\tau_0 \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]$$

On these concepts, Meyer and Peter (1948) has suggested the following formula for calculating the quantity of bed load transport

$$g_b = 0.417\left[\tau_0 \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2} \quad ...(4.51)$$

where g_b is the rate of bed load transport (by wt.) in N per m width of channel per second.

($g_b = q_b \cdot \gamma_w \cdot S_s$)

where q_b = Vol. of sediment transport per metre width of channel per second.

γ_w = Unit wt. of water.

S_s = Sp. gravity of grain.

n' = Manning's coefficient pertaining to grain size on an unrippled bed and given by **Strickler's formula**

i.e. $n' = \frac{1}{24} \cdot d^{1/6}$ where d is the median size (d_{50}) of the bed sediment in metres.

n = the actual observed value of rugosity coefficient on rippled channels. Its value is generally taken as 0.020 for discharges of more than 11 cumecs, and 0.0225 for lower discharges.

τ_c = Critical shear stress required to move the grain in N/m^2, and given by equation (4.45) as

$\tau_c = 0.687\, d_a$;

where τ_c is in N/m^2 and d_a is the mean or average size of the sediment in mm. This arithmetic average size is usually found to vary between d_{50} and d_{60}.

τ_0 = Unit tractive force produced by the flowing water *i.e.* $\gamma_w RS$. Truly speaking, its value should be taken as the unit tractive force produced by the flowing water on bed = $0.97\, \gamma_w RS$.

Example 4.11. *Design a channel which has to carry 25 cumecs with a bed load concentration of 40 p.p.m. by wt. The median grain diameter of the bed material may be taken as 0.3 mm. Use, Lacey's Regime perimeter and Meyer-peter's formulas.*

Solution. Quantity of bed load transported by wt. is 40 parts per million of flowing water

$$= \frac{40}{10^6} \text{ [wt. of flowing water]}$$

Quantity (wt.) of bed load transported per second

$$= \frac{40}{10^6}\left[25 \frac{\text{m}^3}{\text{sec}} \times 9.81 \times 10^3 \frac{\text{N}}{\text{m}^3}\right] = 9.81 \text{ N/sec.}$$

Lacey's Regime perimeter of the channel (P)

$$= 4.75 \cdot \sqrt{Q}$$
$$= 4.75 \cdot \sqrt{25} = 23.75 \text{ m.}$$

Let the channel bed width (B) be kept as 21 m.

Hence, the rate of bed load transport per unit width (by wt.)

$$g_b = \frac{9.81}{21} \text{ N/m/sec.} = \mathbf{0.467 \text{ N/m/sec.}}$$

Meyer Peter's equation (4.51) is

$$g_b = 0.417 \cdot \left[\tau_0 \cdot \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2}$$

where $n' = \frac{1}{24} \cdot d^{1/6}$

where d is median size of sediment in metres $= \frac{0.3}{1000}$ m (given)

$$\therefore \quad n' = \frac{1}{24} \cdot \left(\frac{0.3}{1000}\right)^{1/6} = 0.0108$$

The value of n may be taken as 0.020, because the discharge is more than 11 cumecs or so, and the channel may be taken in good shape and smooth soil.

$$\therefore \quad \frac{n'}{n} = \frac{0.0108}{0.0200} = \mathbf{0.54}$$

From equation (4.45),

$$\tau_c = 0.687 \cdot d_a$$

where d_a is the average particle size in mm ; and we can assume it to be either 0.3 mm or slightly more than that, since mean size is usually found to vary between d_{50} and d_{60}. Let us assume it to be equal to d_{50} *i.e.* 0.3 mm.

$$\therefore \quad \tau_c = 0.687 \times 0.3 = \mathbf{0.206\ N/m^2}$$

Putting these values in Meyer-Peter's formula, we get

$$g_b = 0.417\left[\tau_0 \cdot \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2} \text{ N/m/sec.} \qquad \text{where } \tau_0 = \gamma_w RS$$

$$\therefore \quad 0.467 = 0.417\left[\gamma_w RS \cdot \left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2}$$

or $$0.467 = 0.417\left[(9.81 \times 10^3) \times RS\,(0.54)^{3/2} - 0.206\right]^{3/2}$$

$$0.467 = 0.417 \times (9.81)^{3/2}\left[1000RS\,(0.54)^{3/2} - 0.021\right]^{3/2}$$

or $$\left(\frac{0.467}{0.417 \times (9.81)^{3/2}}\right)^{2/3} = \left[1000\,RS\,(0.54)^{3/2} - 0.021\right]$$

or $$(0.0365)^{0.67} = 1000 \times 0.397\,RS - 0.021$$

or $$0.11 = 397\,RS - 0.021$$

or $$0.131 = 397\,RS$$

or $$\mathbf{RS = 0.00033} \qquad ...(i)$$

Using Manning's equation

$$Q = \frac{1}{n} \cdot AR^{2/3} \cdot S^{1/2} \qquad \text{where } \frac{A}{P} = R \text{ ; or } A = P \cdot R = 23.75 \cdot R$$

$$Q = 25 = \frac{1}{0.020} \times (23.75\,R)\,R^{2/3} \cdot S^{1/2}.$$

or $$\frac{25 \times 0.02}{23.75} = R^{5/3} \cdot S^{1/2}$$

or $$0.021 = R^{5/3} \cdot S^{1/2} \qquad ...(ii)$$

Solving (*i*) and (*ii*), we get

$$RS = 0.00033\,; \quad S = \frac{0.00033}{R}$$

$$\therefore \quad 0.021 = R^{5/3} \times \left(\frac{0.00033}{R}\right)^{1/2}$$

or $$\frac{0.021}{\sqrt{0.00033}} = R^{7/6}$$

$$R = \mathbf{1.13\ m}$$

$$\therefore \quad S = \frac{0.00033}{1.13} \quad \text{or} \quad S = \frac{1}{3424}\,; \text{ say } \mathbf{\frac{1}{3400}}.$$

Let y be the depth of water in the trapezoidal channel of $\frac{1}{2}$: 1 slopes

$$\therefore \quad P = 21 + \sqrt{5} \cdot y$$

$$A = 21y + \frac{y^2}{2}$$

$$R = \frac{A}{P} = \frac{21y + \frac{y^2}{2}}{21 + 2.24 \cdot y} = 1.13 \text{ (worked out earlier)}$$

or $$42y + y^2 = 2.26\,(21 + 2.24\,y)$$

or $$y^2 + 42y = 47.46 + 5.06\,y$$

or $$y^2 + 36.94y - 47.46 = 0$$

or $$y = \frac{-36.94 \pm \sqrt{(36.94)^2 + 4 \times 47.46}}{2} = \frac{-36.94 \pm 39.43}{2}$$

$$= \frac{-36.94 + 39.43}{2} \qquad \text{(using only the feasible + ve sign)}$$

$$= \mathbf{1.25\ m}.$$

Use Depth = 1.25 m.

Hence, use the following channel dimensions.

$$\left.\begin{aligned} B &= 21 \text{ m} \\ y &= 1.25 \text{ m} \\ S &= \frac{1}{3400} \\ \text{Side slopes} &= \tfrac{1}{2}H : 1V. \end{aligned}\right\} \quad \textbf{Ans.}$$

4.9.4. Einstein's Formula (1950). Einstein has put forward a semi-theoretical approach to the problem of bed load transport. In his formula, it is assumed, that every particle after it is dislodged from the bed, travels a certain minimum distance L (which is proportional to the grain diameter) before coming to rest.

Now, let us take a bed area of length L and of unit width. This is clear from the above assumption, that each and every grain that is dislodged from this area will cross the outer boundary (AB) of this area (because each particle has to travel a minimum distance (L). See Fig. 4.15.

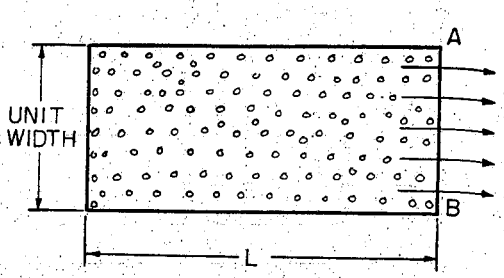


Fig. 4.15.

Now, if $K_1 d^2$ is the bed area covered by each grain (where K_1 is a constant depending upon the shape of the grain), then the number of grains dislodged from this area is given by

$$\frac{L \times 1}{K_1 d^2}$$

If p is the probability of the grain being dislodged in the given second ($\because$ in one particular second, the particles will not be dislodged from the entire area), then the number of particles dislodged per second is given by

$$\frac{L \times 1}{K_1 d^2} \cdot p$$

If $K_2 d^3$ represents the volume of each grain, then the volume of sediment transported per second per unit width is given by

$$q_b = \frac{L \times 1}{K_1 d^2} \cdot p \cdot K_2 \cdot d^3 = \frac{K_2}{K_1} L \cdot p \cdot d.$$

But $L \propto d$ (by assumption made in the beginning)

or $L = K_3 d$

$$\therefore \quad q_b = \frac{K_2}{K_1} K_3 \, p (d)^2 \qquad \ldots(4.52)$$

Einstein further postulated that :

the probability, $p \propto \dfrac{\text{Lift force which the flow can exert on the grain}}{\text{Submerged weight of the grain}}$

But, Lift force $\propto \rho_w V^2 d^2 = \tau_0 d^2$

and Submerged weight of the grain $\propto$ [Submerged density $\times$ volume) $\cdot g$

$$\left[\because \begin{array}{r} \text{Mass} = \text{Volume} \times \text{Density} \\ \text{Weight} = \text{Mass} \times g. \end{array} \right]$$

$\therefore$ Submerged weight of the grain $\propto [\rho_w (S_s - 1) g (K_2 d^3)]$

Using these two values, we get

$$p \propto \frac{\tau_0 d^2}{\rho_w (S_s - 1) g K_2 d^3}$$

or

$$p \propto \frac{\tau_0 d^2}{\rho_w g \cdot (S_s - 1) K_2 d^3}$$

But $\rho_w \cdot g = \gamma_w$.

$$\therefore \quad p \propto \frac{\tau_0 d^2}{\gamma_w (S_s - 1) K_2 \cdot d^3}$$

or $$p \propto \frac{\tau_0}{K_2 \gamma_w (S_s - 1) d}$$

or $$p \propto \frac{\tau_0}{\gamma_w d (S_s - 1)} \qquad ...(4.53)$$

But $\frac{\tau_0}{\gamma_w d (S_s - 1)} = F_s =$ Shield's entrainment function

therefore, $p \propto F_s$.

But F_s is dimensionless and p has the units of time^{-1}, therefore, introducing time, we get

$$p = \frac{1}{t} f(F_s),$$ $f(F_s)$ means, a function of F_s.

Further, it was argued by Einstein that

$$t = \frac{d}{w_0} \quad \textit{i.e.}, \text{ time } (t) = \frac{\text{Distance travelled equal to diameter}}{\text{Fall velocity}}$$

Therefore, $$p = \frac{w_0}{d} f(F_s)$$

or $$p = f(F_s)\left[\frac{w_0}{d}\right] \qquad ...(4.54)$$

Now Eq. (4.52) reduces to

$$q_b = \frac{K_2}{K_1} K_3 \cdot f(F_s)\left[\frac{w_0}{d}\right] \cdot d^2$$

or $$\frac{q_b}{w_0 d} = \frac{K_2}{K_1} K_3 \cdot f(F_s)$$

or $$\frac{q_b}{w_0 d} = \text{F}(F_s) \text{ (some other function of } (F_s) \qquad ...(4.55)$$

Now, $\frac{q_b}{w_0 d}$ is generally represented by ϕ, and F_s is represented by $\frac{1}{\psi}$; where ϕ is known as **Einstein's bed load function**.

Therefore, we get

$$\phi = F\frac{1}{\psi} \qquad ...(4.56)$$

It was also given by Einstein, that

$$w_0 = G\sqrt{gd(S_s - 1)} \qquad ...(4.57)$$

where $$G = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd^3(S_s - 1)}} - \sqrt{\frac{36\nu^2}{gd^3(S_s - 1)}} \qquad ...(4.58)$$

where ν is the kinematic viscosity of water in m^2/sec.

when $d \geq 1.6$ mm, $G \approx \sqrt{(2/3)} \approx 0.81$

The above ϕ–ψ relationship is represented by the curve, shown in Fig. 4.16, and can be used to compute bed load transport rate of a given channel; or conversely to design stable channels for a given bed load transport.

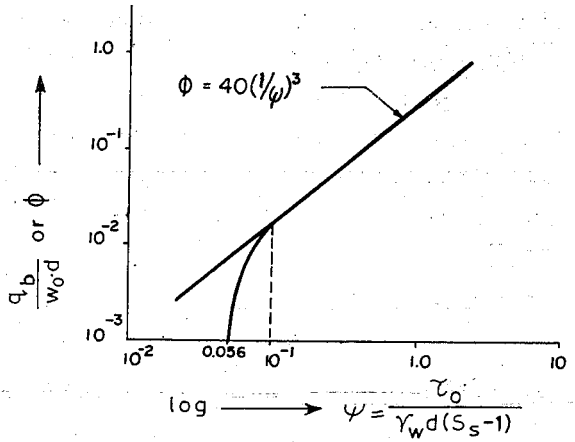


Fig. 4.16.

The straight line portion of this curve, follows the relation

$$\phi = 40 \cdot \left(\frac{1}{\psi}\right)^3 \qquad ...(4.59)$$

or
$$\boxed{\frac{q_b}{w_0 d} = 40 \cdot \left[\frac{\tau_0}{\gamma_w d\,(S_s - 1)}\right]^3} \qquad ...(4.60)$$

Now, $w_0 = G \cdot \sqrt{gd\,(S_s - 1)}$

$$\therefore \quad \frac{q_b}{G \cdot d\sqrt{gd\,(S_s-1)}} = 40\left[\frac{\tau_0}{\gamma_w d\,(S_s-1)}\right]^3 \qquad ...(4.61)$$

But $\tau_0 = \gamma_w RS$.

$$\therefore \quad \frac{q_b}{G \cdot d\sqrt{gd\,(S_s-1)}} = 40\left[\frac{\gamma_w RS}{\gamma_w \cdot d\,(S_s-1)}\right]^3$$

or
$$\frac{q_b}{G \cdot d\sqrt{gd\,(S_s-1)}} = 40\left[\frac{R^3 \cdot S^3}{d^3\,(S_s-1)^3}\right]$$

or
$$q_b \propto \frac{R^3 \cdot S^3 \cdot d\sqrt{d}}{d^3}$$

or
$$q_b \propto \frac{R^3 S^3}{d^{3/2}} \qquad ...(4.62)$$

Now, if it is assumed that the channel is wide and Chezy's C is constant, then

$$Q = C \cdot A\sqrt{RS}$$

$$\therefore \quad q = C \cdot y \cdot \sqrt{RS} \qquad \text{But } R \approx y \text{ for wide channels or rivers.}$$

$$\therefore \quad q = C \cdot R \cdot \sqrt{RS} = C \cdot R^{3/2} S^{1/2}$$

or
$$q^2 \propto R^3 \cdot S \qquad ...(4.63)$$

From Eq. (4.62), $q_b \propto \frac{R^3 \cdot S \cdot S^2}{d^{3/2}}$

or $$q_b \propto \frac{q^2 \cdot S^2}{d^{3/2}} \quad ...(4.64)$$

or $$\boxed{\frac{q_b}{q} \propto \frac{q \cdot S^2}{d^{3/2}}} \quad ...(4.65)$$

This is an important form of Einstein's equation, and shows that the sediment carrying capacity of the channel depends upon the discharge per unit width. This relationship helps us to draw the following very interesting and important conclusions:

(1) If the width of a river increases, the discharge per unit width (q) will decrease (Q remaining constant) and, therefore, q_b/q *i.e.*, sediment carrying capacity will reduce. Hence, the deposition of the sediment will start, which will increase the bed slope. The deposition will continue till the slope S is increased to a value sufficient to make qS^2 a constant.

(2) Since the sediment carrying capacity depends upon the discharge, the floods will carry more sediment. In fact, the floods are mainly responsible for most of the annual sediment load.

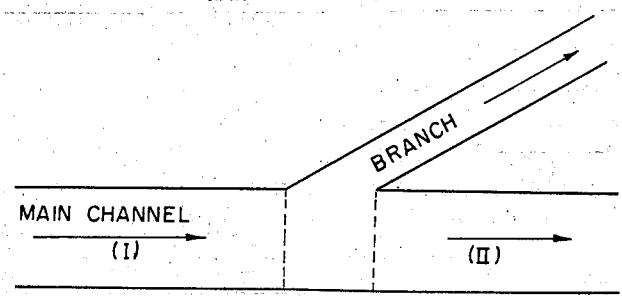


Fig. 4.17.

(3) If a branch channel starts from a main channel, then an interesting phenomenon may happen, as described below :

Let a branch channel off-takes from a main channel, as shown in Fig. 4.17.

Now, after the branch channel starts extracting some discharge, the discharge in the main channel reduces, thereby reducing its sediment carrying capacity. Hence, deposition of sediment will start, which further diverts the flow towards the off-taking channel, causing more reduction in discharge in the main channel and more silt deposition ; causing more inflow towards the branch channel. The process may continue till the entire water gets diverted towards the branch channel.

The prime cause for this phenomenon is the existence of a favourable gradient in the branch channel, but the mechanism which keeps the process going, is necessarily, what is explained above by the Einstein's equation.

A typical example exhibiting the importance of this phenomenon is quoted below:

Mississippi river is connected to another old river Atchafalya by a branch channel, which off-takes from the Mississippi river at about 480 kilometres from its mouth, as shown in Fig. 4.18.

The branch channel has been capturing flow from the main channel for the last many decades. In fact, engineers have tried to promote this flow as a means of diverting flood waters. About 26% of discharge was flowing towards this channel at one stage, with

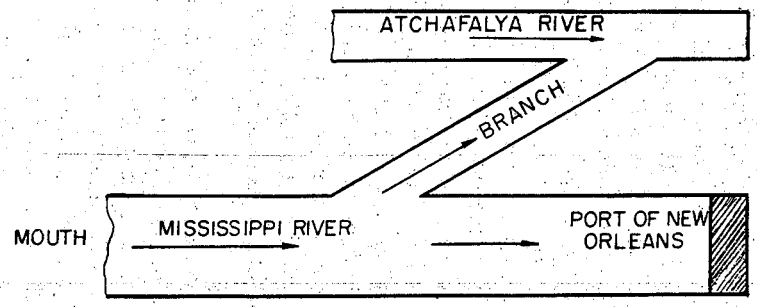


Fig. 4.18.

tendency of further increase. But, if the proportion is allowed to become rather higher and reaches to a value of the order of 40% or so, the process would become almost irreversible, and the whole flow may be rapidly diverted towards the Atchafalya river through this branch channel. If such a state arises, the port of New Orleans (situated at the join of Mississippi river with sea) could become redundant.

Example 4.12. *Determine the rate of bed load transport in a wide alluvial stream for the following data :*

Depth of flow = 4.5 m

Velocity of flow = 1.3 m/s

Bed slope $= 2.0 \times 10^{-4}$

Size distribution of the sediment :

d (mm)	*0.20*	*0.44*	*0.78*	*1.14*	*1.65*	*3.6*	*5.2*
% finer	*2*	*10*	*30*	*50*	*70*	*80*	*100*

Make use of Einstein's formula for bed load. Kinematic viscosity of water $= 1.01 \times 10^{-6}\ m^2/s$.

Solution. $V = \frac{1}{n} \cdot R^{2/3} \cdot \sqrt{S}$

where $R \approx D$ (for wide streams) = 4.5 m (given)

$V = 1.3$ m/s (given)

$S = 2 \times 10^{-4}$ (given)

$\therefore \quad 1.3 = \frac{1}{n} \cdot (4.5)^{2/3} \cdot \sqrt{2 \times 10^{-4}}$

or $\quad n = \mathbf{0.02965}$

Also, $\quad n' = \frac{1}{24} \cdot (d_{50})^{1/6}$

where $d_{50} = 1.14$ mm from the given table $= 1.14 \times 10^{-3}$ m

$$= \frac{1}{24} \cdot (1.14 \times 10^{-3})^{1/6} = \mathbf{0.0135}$$

$$\frac{n'}{n} = \frac{0.0135}{0.02965} \mathbf{0.454}$$

$$R' = R \cdot \left(\frac{n'}{n}\right)^{3/2} = 4.5\ (0.454)^{3/2} = \mathbf{1.377\ m}$$

Using equation (4.57), we have

$$w_0 = \text{fall velocity} = G \cdot \sqrt{gd\,(S_s - 1)}$$

$$\text{where } G = \sqrt{\frac{2}{3} + \frac{36\nu^2}{gd^3\,(S_s - 1)}} - \sqrt{\frac{36\,\nu^2}{gd^3\,(S_s - 1)}}$$

$$\text{where, } \frac{36\,\nu^2}{gd^3\,(S_s - 1)} = \frac{36 \times (1.01 \times 10^{-6})^2}{9.81 \times (1.14 \times 10^{-3})^3 \cdot 1.65}$$

$$= 1.5314 \times 10^{-3}$$

$$G = \sqrt{0.6667 + 1.5314 \times 10^{-3}} - \sqrt{1.5314 \times 10^{-3}}$$

$$= 0.817 - 0.039 = 0.778$$

$$\therefore \quad w_0 = 0.778\sqrt{9.81 \times (1.14 \times 10^{-3})\,(1.65)} = \mathbf{0.105\ m/s}$$

Using equation (4.60), we have

$$\frac{q_b}{w_0 \cdot d} = 40\left[\frac{\gamma_w R'S}{\gamma_w d\,(S_s - 1)}\right] = 40\left[\frac{R'S}{d\,(S_s - 1)}\right]^3 \qquad \because\ \tau_0 = \gamma_w R'S$$

To appreciate bed ripples, R' is used in place of R

$$\text{or} \quad \frac{q_b}{0.105 \times 1.14 \times 10^{-3}} = 40\left[\frac{1.377 \times 2.0 \times 10^{-4}}{1.14 \times 10^{-3}\,(1.65)}\right]^3$$

$\therefore$ Bed load by vol.

$$= q_b = 1.5027 \times 10^{-5}\ \text{m}^3/\text{sec/m width of channel.}$$

Bed load by weight (g_b)

$$g_b = q_b \cdot \gamma_w \cdot S_s$$

$$= 1.5027 \times 10^{-5} \times (9.81 \times 10^3) \times 2.65\ \text{N/sec/m} = \mathbf{0.39\ N/s/m} \quad \textbf{Ans.}$$

Example 4.13. *A wide irrigation channel is designed to have hydraulic mean depth of 3 m and bed slope equal to 1.6×10^{-4}. The bed sediment has an average median size of 0.3 mm. If the specific gravity of the bed soil is taken as 2.65 and the observed Manning's n to be 0.020, compute the rate of bed load transported by the channel in N/s/m width of channel.*

Also compute the suspended load concentrations at different depths to plot the sediment concentration curve for this channel. Make use of Einstein's formulas. Kinematic viscosity of water at given temperature may be taken to be $1.01 \times 10^{-6}\ m^2/s$.

Solution. Bed load transport rate as volume (q_b) is given by Einstein's equation (4.60) as :

$$\frac{q_b}{w_0 \cdot d} = 40\left[\frac{\gamma_w R'S}{\gamma_w \cdot d\,(S_s - 1)}\right]^3 = 40 \cdot \left[\frac{R'S}{d\,(S_s - 1)}\right]^3$$

To appreciate bed ripples, R' is used in place of R.

$$\text{where } w_0 = G \cdot \sqrt{gd\,(S_s - 1)} \qquad \ldots(4.57)$$

$$\text{where } G = \sqrt{\frac{2}{3} + \frac{36 \cdot \nu^2}{gd^3\,(S_s - 1)}} - \sqrt{\frac{36\,\nu^2}{gd^3\,(S_s - 1)}}$$

where ν = kinematic viscosity of water at 20° C = 1.01×10^{-6}.

$$\frac{36\,\nu^2}{gd^3\,(S_s-1)} = \frac{36\times(1.01\times10^{-6})^2}{9.81\times(0.3\times10^{-3})^3\,(1.65)} = \frac{36\times1.01\times1.01\times10^{-12}}{9.81\times(0.3)^3\,(1.65)\times10^{-9}} = 0.084$$

$$\therefore \quad G = \sqrt{0.667+0.084} - \sqrt{0.084} = 0.867 - 0.290 = \mathbf{0.577}$$

$$w_0 = 0.577\sqrt{9.81\times(0.3\times10^{-3})\,(1.65)} = \mathbf{0.04\ m/sec.}$$

Using equation (4.60) :

$$\frac{q_b}{0.04\times(0.3\times10^{-3})} = 40\left[\frac{R'S}{d\,(S_s-1)}\right]^3$$

where $R' = R\left(\frac{n'}{n}\right)^{3/2}$

where $n' = \frac{1}{24}\cdot d^{1/6} = \frac{1}{24}(0.3\times10^{-3})^{1/6} = 0.01078$

$n = 0.020$ (given)

$$\frac{n'}{n} = \frac{0.01078}{0.020} = 0.539$$

$R = 3$ m (given)

$$R' = 3\times(0.539)^{3/2} = \mathbf{1.187\ m}$$

$$\therefore \quad \frac{q_b}{0.040\times(0.3\times10^{-3})} = 40\left[\frac{1.187\times(1.6\times10^{-4})}{(0.3\times10^{-3})\,(1.65)}\right]^3 = 40\,(0.3837)^3 = 2.259$$

or $q_b = (2.259\times0.040\times0.3\times10^{-3})$

$= \mathbf{2.711\times10^{-5}\ m^3/sec/m}$ width of channel

$g_b = q_b\cdot\gamma_w\cdot S_s$

$$= 2.711\times10^{-5}\,\frac{\text{m}^3}{\text{sec.m}}\times\left(9.81\times10^3\,\frac{\text{N}}{\text{m}^3}\right)\times2.65$$

$= 0.7$ N/s/m width of channel.

Hence, bed load transported per m width of channel

$= \mathbf{0.7\ N/s/m\ width.\ Ans.}$

For computing suspended load, we compute from Eq. (4.43),

$$c_a = c_{2d} = \frac{q_b}{23.2\times V^{*\prime}\times d}\times100\%$$

or $$c_a = \frac{2.711\times10^{-5}}{23.2\times V^{*\prime}\times(0.3\times10^{-3})}\times100\%$$

where $V^{*\prime} = \sqrt{\frac{\tau_0'}{\rho_w}} = \sqrt{\frac{\rho_w g R'S}{\rho_w}} = \sqrt{gR'S}$

$= \sqrt{9.81\times1.187\times1.6\times10^{-4}} = \mathbf{0.043\ m/sec}$

$$\therefore \quad c_a = \frac{2.711\times10^{-5}}{23.2\times0.043\times(0.3\times10^{-3})}\times100\% = \mathbf{9.058\% = 90{,}580\ ppm.}$$

Now, using Eq. (4.40), we have

$$\frac{c}{c_a} = \left[\frac{a\,(D-y)}{y\,(D-a)}\right]^{\frac{w_0}{KV^*}}$$

where $V^* = \sqrt{gRS}$

$= \sqrt{9.81 \times 3 \times 1.6 \times 10^{-4}} = 0.069$ m/sec

$K = 0.4$ (constant)

$a = 2d = 2 \times 0.3 \times 10^{-3}$ m

$$\therefore \quad \frac{w_0}{KV^*} = \frac{0.04}{0.4 \times 0.069} = 1.457$$

$$\therefore \quad \frac{c}{9.058\%} = \left[\frac{(2 \times 0.3 \times 10^{-3})(3-y)}{y\,(3 - 2 \times 0.3 \times 10^{-3})}\right]^{1.457} = 2 \times 10^{-4}\left[\frac{3-y}{y}\right]^{1.457}$$

or

$$c = 1.812 \times 10^{-3}\left(\frac{3-y}{y}\right)^{1.457}$$

The different values of c at different values of y, say 0.05 m, 0.1 m, 0.15 m, 0.2 m, 0.5 m, 1.0 m, 2.0 m, 3.0 m are now computed by using the above equation, as :

$$c_{0.05m} = 1.812 \times 10^{-3}\left(\frac{3-0.05}{0.05}\right)^{1.457} = 0.6891\% = 6891 \text{ ppm}$$

$$c_{0.1\,m} = 1.812 \times 10^{-3}\left(\frac{3-0.1}{0.1}\right)^{1.457} = 0.2447\% = 2447 \text{ ppm}$$

$$c_{0.15\,m} = 1.812 \times 10^{-3}\left(\frac{3-0.15}{0.15}\right)^{1.457} = 0.1264\% = 1264 \text{ ppm}$$

$$c_{0.2\,m} = 1.812 \times 10^{-3}\left(\frac{3-0.2}{0.2}\right)^{1.457} = 0.0847\% = 847 \text{ ppm}$$

$$c_{0.5\,m} = 1.812 \times 10^{-3}\left(\frac{3-0.5}{0.5}\right)^{1.457} = 0.0188\% = 188 \text{ ppm}$$

$$c_{1.0\,m} = 1.812 \times 10^{-3}\left(\frac{3-1.0}{1.0}\right)^{1.457} = 0.0050\% = 50 \text{ ppm}$$

$$c_{2.0\,m} = 1.812 \times 10^{-3}\left(\frac{3-2.0}{2.0}\right)^{1.457} = 0.0005\% = 5 \text{ ppm}$$

$$c_{2.5\,m} = 1.812 \times 10^{-3}\left(\frac{3-2.5}{2.5}\right)^{1.457} = 0.00025\% = 2.5 \text{ ppm}$$

$$c_{3\,m} = 1.812 \times 10^{-3}\left(\frac{3-3}{3}\right)^{1.457} = \text{zero.}$$

DESIGN PROCEDURE FOR IRRIGATION CHANNELS

After having discussed the various theories for designing irrigation canals, we shall now discuss, as to how in actual practice, an irrigation canal is designed and constructed.

4.10. Cross-section of an Irrigation Canal

A typical and most desired section of a canal is shown in Fig. 4.19. This section is 'partly in cutting and partly in filling', and aims at balancing the quantity of earth work in 'excavation' with that in 'filling'.

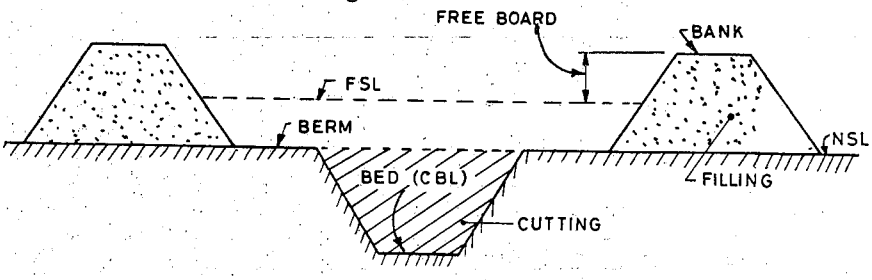


Fig. 4.19.

Sometimes, when the natural surface level (*i.e. NSL*) is above the top of the bank, the entire canal section will have to be in cutting, and it shall be called 'canal in cutting'. Similarly, when the *NSL* is lower than the Bed level of the canal, the entire canal section will have to be built in filling, and it is called 'canal in filling', or 'canal in banking'.

Side Slopes. The side slopes should be such that they are stable, depending upon the type of the soil. A comparatively steeper slope can be provided in cutting rather than in filling, as the soil in the former case shall be more stable.

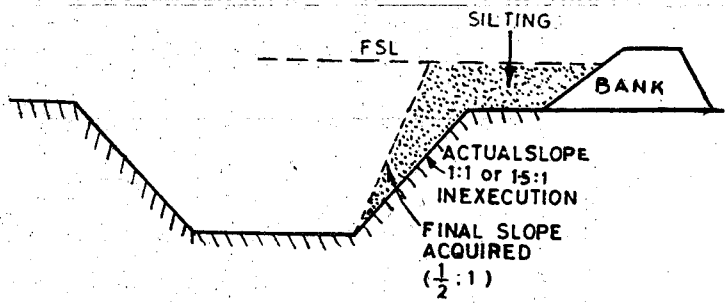


Fig. 4.20.

$1H : 1V$ (1 : 1) to $1\frac{1}{2}H : 1V$ ($1\frac{1}{2}$: 1) slope in cutting, and $1\frac{1}{2}H : 1V$ to $2H : 1V$ in filling, are generally adopted.

In case of channels with silt laiden water, the actual capacity of the channel is worked out with $\frac{1}{2}$: 1 side slopes, even though flatter slopes such as 1 : 1 ot $1\frac{1}{2}$: 1 may be constructed at the time of execution. This is because of the fact that the sides of such a channel gets silted up to a slope $\frac{1}{2}$: 1 with the passage of time, as shown in Fig. 4.20.

Berms. Berm is the horizontal distance left at ground level between the toe of the bank and the top edge of cutting.

The berm is provided in such a way that the bed line and the bank line remains parallel. If $s_1 : 1$ is the slope in cutting and $s_2 : 1$ in filling, then the initial berm width $= (s_2 - s_1)\, d_1$. Since ground level (NSL) fluctuates considerably, while canal bed level (CBL) varies very slightly, d_1 shall vary ; and, therefore, the berm width shall vary.

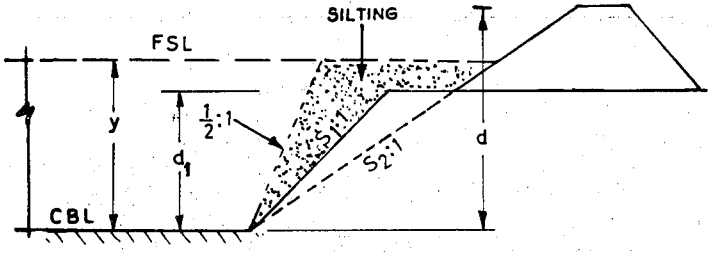


Fig. 4.21.

After the water flows in the channel for some time, the silt gets deposited on the sides giving them a slope of $\frac{1}{2} : 1$. The position of the berm, therefore, changes from ground level to FSL, as shown in Fig. 4.21 and its width becomes equal to $(s_2 - \frac{1}{2}) \cdot y$. If $s_2 = 1\frac{1}{2}$, then the final berm width = y, *i.e.* equal to the depth of the canal.

The berms when fully formed, serve the following purposes :

(*i*) The silt deposited on the sides is very fine and impervious. It, therefore, serves as a good lining for reducing losses, leakage and consequent breaches, etc.

(*ii*) They help the channel to attain regime conditions, as they help in providing a wider waterway, if required. Even fluctuations of discharge do not produce much fluctuations in depths because of wider waterway.

(*iii*) They give additional strength to the banks and provide protection against erosion and breaches.

(*iv*) The possibility of breaches gets reduced because the saturation line comes more in the body of the embankment.

(*v*) They protect the banks from erosion due to wave action.

(*vi*) They provide a scope for future widening of the canal.

(*vii*) Berms can be used as borrow pits for excavating soil to be used for filling.

Freeboard. The margin between FSL and bank level is known as freeboard. The amount of freeboard depends upon the size of the channel. The generally provided values of freeboard are given in Table 4.7.

Table 4.7. Values of Free-boards in Canals

Discharge in cumecs	*Extent of freeboard in metres*
1 to 5	0.50
5 to 10	0.60
10 to 30	0.75
30 to 150	0.90

Banks. The primary purpose of banks is to retain water. They can be used as means of communication and as inspection paths. They should be wide enough, so that a minimum cover of 0.5 metre is available above the saturation line, as shown in Fig. 4.22. High banks will have to be designed as earth dams.

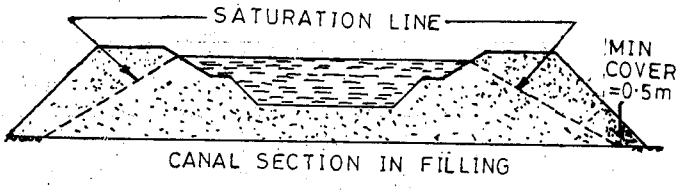


Fig. 4.22.

Service Roads. Service roads are provided on canals for inspection purposes, and may simultaneously serve as the means of communication in remote areas. They are provided 0.4 m to 1.0 m above FSL, depending upon the size of the channel.

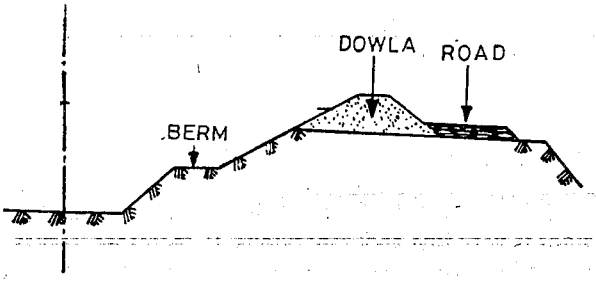


Fig. 4.23.

Dowlas. As a measure of safety in driving, dowlas 0.3 m high and 0.3 to 0.6 m wide at top, with side slopes of $1\frac{1}{2}:1$ to $2:1$, are provided along the banks, as shown in Fig. 4.23. They also help in preventing slope erosion due to rains, etc.

Back Berm or Counter Berms. Even after providing sufficient section for bank embankment, the saturation gradient line may cut the downstream end of the bank. In such a case, the saturation line can be kept covered at least by 0.5 metre with the help of counter berms, as shown in Fig. 4.24.

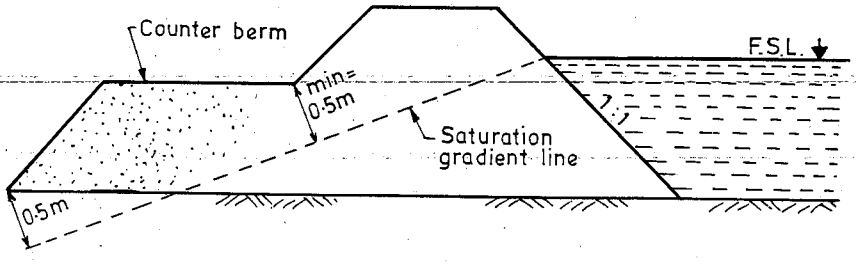


Fig. 4.24.

The straight saturation gradient line may be drawn with the following slopes.

Table 4.8. Assumed Values of Saturation Gradients in Different Soils

Type of soil	*Slope (H : V)*
Clay	1 in 4
Clayey Loam	1 in 6
Loam	1 in 8
Loamy sand	1 in 10
Sand	1 in 15

Spoil Banks. When the earthwork in excavation exceeds earthwork in filling, even after providing maximum width of bank embankments, the extra earth has to be disposed of economically. To dispose of this earth by mechanical transport, etc. may become very costly, and an economical mode of its disposal may be found in the form of collecting this soil on the edge of the bank embankment itself. The soil is, therefore, deposited in such a case, in the form of heaps on both banks or only on one bank, as shown in Fig. 4.25. These heaps of soil are discontinued at suitable intervals and longitudinal drains running by their sides are excavated for the disposal of rain water. Cross drains through the spoil banks may also be excavated, if needed.

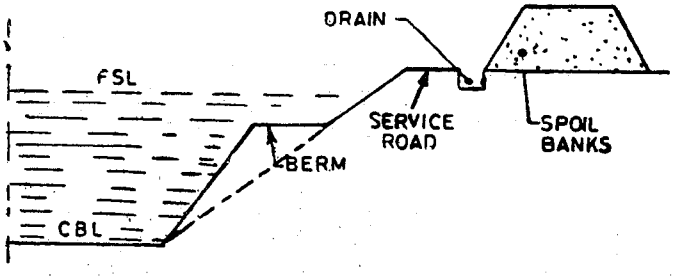
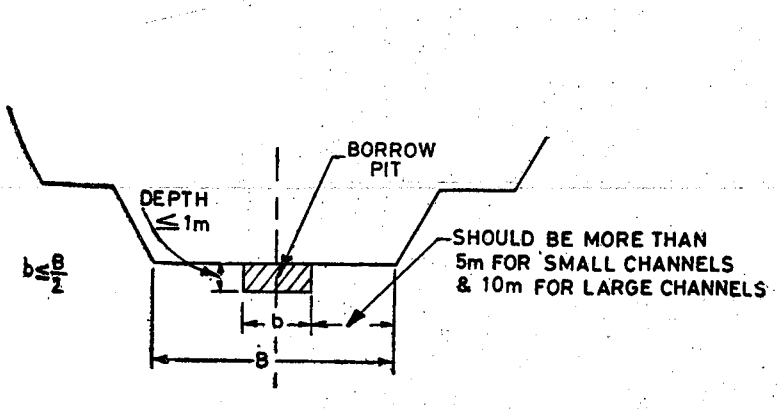
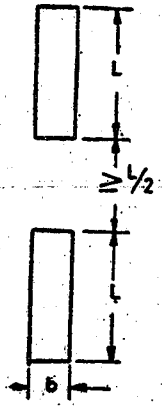


Fig. 4.25. Spoil bank.

Borrow Pits. When earthwork in filling exceeds the earthwork in excavation, the earth has to be brought from somewhere. The pits, which are dug for bringing earth, are known as borrow pits. If such pits are excavated outside the channel, they are known as *external borrow pits,* and if they are excavated somewhere within the channel, they



(*a*). Elevation.



(*b*). Plan.

Fig. 4.26. Internal borrow pits.

are known as *internal borrow pits.* It is a very costly affair to bring soil from distances. Even in the nearby areas, these pits may cause mosquito nuisance due to collection of rain water in these pits, and hence, external borrow pits are not preferred.

Internal borrow pits are, therefore, excavated on the bed of the canal, as shown in Fig. 4.26, when needed.

The borrow pits should start from a point at a distance more than 5 m from the toe for small channels, and 10 m for large channels. The width of these pits b, should be less than half the width of the canal B, and should be dug in the centre. The depth of these pits should be equal to or less than 1 m.

Longitudinally, these pits should not run continuous, but a minimum space of $\frac{L}{2}$ should be left between two consecutive pits, (where L is the length of one pit) as shown in Fig. 4.26 (b).

4.11. Balancing Depth for Excavating Canals

It was pointed out earlier that the maximum economy can be achieved in canal construction, if the earthwork in excavation equals the earthwork in filling. Such a thing is possible when a canal is constructed partly in filling and partly in cutting, which mostly happens in practical life.

If this balance between cutting and filling can occur, then the need for spoil banks or borrow pits is entirely eliminated, and moreover, earthwork has to be paid only once in a single item.

For a given cross-section of a channel, there can be only one depth, for which such a balance between cutting and filling will occur. This depth is known as the Balancing depth. This depth can be worked out easily by equating the areas of cutting and filling, as illustrated in the following example.

Example 4.14. *Calculate the balancing depth for a channel section having a bed width equal to 18 m and side slopes of 1 : 1 in cutting and 2 : 1 in filling. The bank embankments are kept 3.0 m higher than the ground level (berm level) and crest width of banks is kept as 2.0 m.*

Solution. The channel section is shown in Fig. 4.27. Let d_1 be the balancing depth, *i.e.* the depth for which excavation and filling becomes equal.

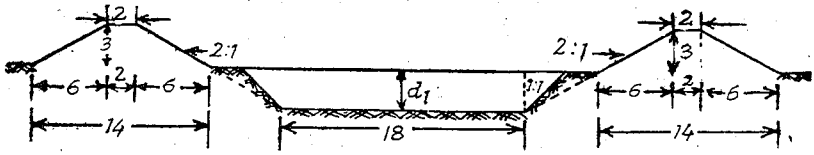


Fig. 4.27.

Area of cutting $= (18 + d_1)\, d_1$ sq. m.

Area of filling $= 2\left[\frac{2+14}{2} \times 3\right] = 48$ sq. m.

Equating cutting and filling, we get

$$(18 + d_1)\, d_1 = 48 \qquad \text{or} \qquad 18d_1 + d_1^2 = 48$$

or $$d_1^2 + 18d_1 - 48 = 0$$

or $$d_1 = \frac{-18 \pm \sqrt{324 + 192}}{2}$$

$$= \frac{-18 \pm 22.7}{2}$$

Ignoring unfeasible – ve sign, we get

$$d_1 = \frac{-18 + 22.7}{2} = \frac{4.7}{2} = 2.35 \text{ m}$$

$\therefore$ Balancing Depth = **2.35 m. Ans.**

Example 4.15. *An irrigation channel having a full supply level of 4 m above the existing ground level is provided with banks 3 m wide at top. The side slopes are 2H : 1 V ; and the slope of the hydraulic gradient line through the bank soil is 5 : 1. Assuming a free board of 1 m, calculate the minimum width and height of counterberm needed to ensure that the seepage of water does not pose any problem for the safety of the canal banks.*

Solution. The given values are used in sketching the *X*-section of the Canal, as shown Fig. 4.28. The H.G. line is kept covered by 0.5 m (see $EF = HI = 0.5$ m).

The counter berm is started from the point E on the D/s slope, at a height h metre above the bed level. The width of the counter berm is kept as equal to EG ; say B metre. *Now, From H.G. line slope,* we have

$$NJ = 4 \times 5 = 20 \text{ m}$$

(1 : 5 slope of H.G. line starting at ht. = 4 m)

But $$NJ = NM + ML + LK + KJ$$

or $$20 = 2 + 3 + 10 + KJ$$

$\therefore$ $$KJ = 5 \text{ m}.$$

But $$B = EG = KH \text{ (In } \| \text{ gm } EKHG)$$

or $$B = KJ + JH$$

or $$B = 5 + JH \qquad ...(i)$$

where JH is determined from ΔHIJ ; as :

$$\frac{0.5}{JH} = \sin \alpha$$

where α is the angle which H.G. line makes with horizontal ; *i.e.*, $\tan \alpha = \frac{1}{5}$; or $\alpha = 11.31°$

$$\therefore \quad \frac{0.5}{JH} = \sin 11.31°$$

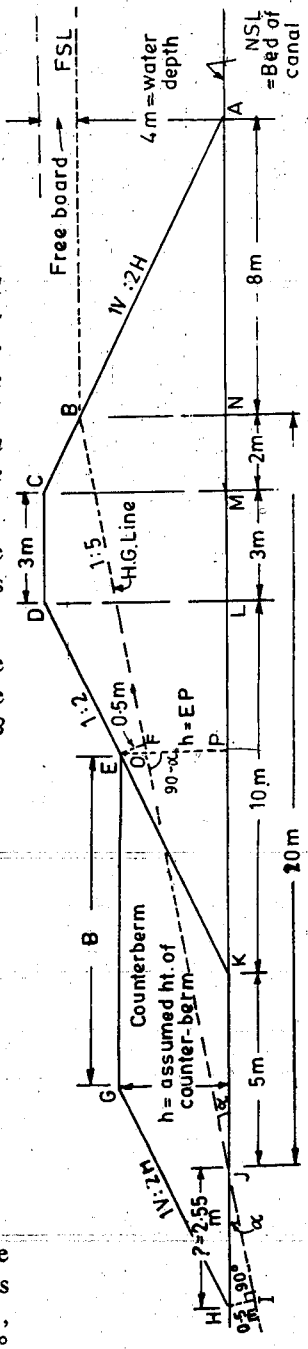


Fig. 4.28

$$\text{or } JH = \frac{0.5}{\sin 11.31°} = \mathbf{2.55\ m} \qquad ...(ii)$$

$\therefore$ $B = 5 + 2.55 = \mathbf{7.55\ m}$ **Ans.**

To determine ht of counter berm (h), we calculate as below :

$$h = EP = EO + OP$$

where EO is calculated from ΔEOF as

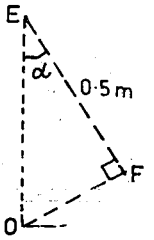


Fig. 4.28 (*a*)

$$\frac{EF}{EO} = \frac{0.5}{EO} = \cos\alpha$$

$$\text{or } EO = \frac{0.5}{\cos\alpha} = \frac{0.5}{\cos 11.35°} = 0.51\ \text{m}$$

$\therefore$ $OP = h - 0.51$ ...(*iii*)

But $\frac{OP}{PJ} = \frac{1}{5}$ (slope of H.G. line)

$$\text{or } \frac{h - 0.51}{PJ} = \frac{1}{5} \qquad ...(iv)$$

But $PJ = PK + KJ$

$= 2h + 5$ ...(*v*)

Putting in (*iv*)

$$\frac{h - 0.51}{2h + 5} = \frac{1}{5}$$

$$5h - 2.55 = 2h + 5$$

$$3h = 7.55$$

$$h = \mathbf{2.52\ m.\ Ans.}$$

Hence, width of counter berm = 7.55 m
ht. of counterberm = 2.52 m **Ans.**

4.12. Fixing the L-Section of the Canal and Other Design Considerations

If a channel is designed according to Lacey's theory, it shall have a fixed slope and a fixed section for a given discharge and silt factor. But on the other hand, if the channel is designed on Kennedy's theory, it can have different sections for different slopes. In practice, it has been found that Lacey's slope equation gives excessive slopes. Lacey himself had changed his fixed slope equation afterwards, as pointed out earlier, making it flexible. *The slope of the channel is, however, fixed on available country slope consistent with economy.* A steeper slope governed by maximum permissible velocity, will be most economical, but it will lower the FSL, causing less irrigation. Hence, the maximum possible irrigation would indicate flatter slopes governed by minimum permissible velocity. A via media between these two limits must be adopted for selecting a suitable bed slope for the channel.

If the chosen designed slope is found to be flatter than the natural available slope, the difference can be adjusted by providing suitably designed falls (explained in chapter 12). But if the designed slope is steeper than that available, then adjustments are made to change the design slope, so as to make it as near to the available slope as possible.

Since a change in depth causes non-uniform flow, it is desirable to change the depth as less as possible. For this reason, the channels in the upper reaches are generally designed with large bed width to depth ratio.

Moreover, in Kennedy's theory, there can be various combinations of width and depth for a given slope. The width and depth ratio can be controlled using the following empirical formulas ;

1. (*a*) *For channels upto 15 cumecs*

$$y = 0.5\sqrt{B} \qquad \ldots(4.62)$$

where y = Depth of water in channel

B = Base width of channel.

1. (*b*) For channels of 15 cumecs and above, depths of the following order may be provided :-

Table 4.9. Assumed Values of Water Depths for Canals

Discharge Q (cumecs)	*Depth y in metres*
15	1.7
30	1.8
75	2.3
150	2.6
300	3.0

2. C.W.C. has recommended a graphical relation giving B/y ratios for various discharges (0 to 300 cumecs) as shown in Fig. 4.29.

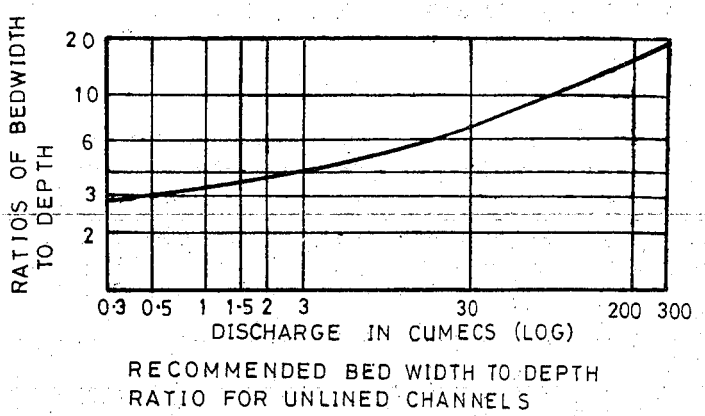


Fig. 4.29.

The limiting maximum permissible velocities for unlined channels in different soils are given in Table 4.10.

Table 4.10. Max. Permissible Velocities in Unlined Channels

Type of soil	*Maximum permissible velocity in m/sec*
Rock and gravel	1.5
Murum, hard soil, etc.	1.0 to 1.1
Sandy loam, black cotton soil etc.	0.6 to 0.9
Very light loose sand to average sandy soils	0.3 to 0.6
Ordinary soils	0.6 to 0.9

After marking the ground line, a trial bed line, and water depths, the FSL line on the *L*-section of the channel is checked, as it is governed by the following considerations:

(*i*) The FSL of the channel should be about 10 to 20 cm above the ground line for most of its length.

(*ii*) The water depth (y) or FSL governs the depth of excavation (d_1) and hence, attempts should be made to equal d_1 with balancing depth, so that the channel becomes in balanced earth work.

(*iii*) For branch canal, etc., where some distributaries have to take-off from it, the FSL of the channel should be kept 15 cm higher than the FSL of the off-taking channel, so as to allow for losses in head regulator.

Suitable adjustments in bed slope, depth, etc. can be made for fixing FSL on these considerations.

After fixing *L*-section of the channel, the cross-section can be fixed on the basis of various canal standards, given in Table 4.11.

Design Procedure

1. First of all, the longitudinal section of the existing ground along the proposed canal alignment, is plotted on a suitable scale (say 1 cm = 100 metres for horizontal scale, and 2 cm = 1 m for vertical scale).

2. A suitable channel slope is assumed from Lacey's diagrams for an approximate discharge and silt factor. It is made consistent with the existing slope.

3. A trial slope line is marked for drawing FSL line, keeping in view the guidelines already given. The depth and position of falls, etc. is decided tentatively.

4. The channel is then designed from its tail reach towards its head reach, kilometre to kilometre.

5. The discharge required in the channel in the given reach, for required irrigation potential, is worked out, and losses are added, so as to calculate the required discharge. The cumulative losses at a particular km of channel is the sum total of the losses occurring between the particular km and the next below it (*i.e.* for example at km 3, the total losses would be equal to losses between km 3 and km 4, plus, total losses below km 4).

Since the section at km 4 will be worked out earlier, the losses between km 3 and km 4 can be worked out on the wetted perimeter calculated on the basis of the channel section at km 4. The required discharge is increased by 10%, so as to obtain the design discharge.

6. The channel is now designed for this discharge and assumed bed slope, by Kennedy's theory, using Garret's diagrams. The bed width and depth ratios are kept within specified limits. Sometimes, even the slope may have to be changed for keeping them so.

7. The bed slope, FSL, falls, etc. are all adjusted using intelligence, judgement and knowledge. The bed levels, water depths, etc. are drawn on *L*-section. The *X*-sections at every km are then drawn, using canal standards.

Table 4.11. Canal Standards

S. No.	Item	Size of canal					
		Minor Distributaries		Major Distributaries		Main and Branch Canals	
		below 0.3 cumec	0.3 to 1 cumec	1 to 5 cumecs	5 to 10 cumecs	10 to 30 cumecs	30 to 150 cumecs
1.	Maximum width of the bank crest	1.0 m	1.5 m	2 m	2.25 m	2.5 m	3 m
2.	Width of roadway	Nil	3.5 m	3.5 m	5 m	5 m	6 m
3.	Free board	0.3 m	0.4 m	0.5 m	0.6 m	0.75 m	0.9 m
4.	Depth of earth cover over saturation gradient	0.5 m	0.5 m	0.5 m	0.5 m	0.8 m	1 m
5.	Width of berms	Maximum berm width = 0.6 m+ $\frac{1}{4}$th of the width of combined side slopes of cutting and embankment. Maximum berm width = 0.6 m+ $\frac{1}{2}$ the width of combined slopes					
6.	Width of land to be acquired (clear of banks) when canal cutting is deeper than the balancing depth.	Half the height of the bank above ground, subject to minimum of 1.5 m.					
7.	Width of land to be acquired (clear of banks) when canal cutting is lesser than the balancing depth.	Full height of bank above ground level plus 1.5.				Full height of bank+ 5 m.	Full height of bank+ 5 m

The calculations of discharge and channel dimensions, etc. can be carried out in a tabular form called "Schedule of Area Statistics and Channel Dimensions'. A specimen form of this table is given in table 4.12.

Tabel 4.12. Specimen Table for Schedule of Area Statistics and Channel Dimensions

Below km	*Gross commanded area*	*Culturable commanded area*	*Area to be irrigated*		
			Rabi	*Sugar*	*Rice*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
Outlet discharge factor	*Outlet discharge*	*Losses in reach*	*Total losses*	*Total discharge*	*Bed slope*
(7)	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*
Bed width	*Water depth*	*Height of banks*	*Width of banks*	*Velocity*	*CVR* V/V_0
(13)	*(4)*	*(15)*	*(16)*	*(17)*	*(18)*

Example 4.16. *A distributary canal takes off from a branch canal having CBL at 204.0 m and FSL at 205.8 m. The gross commanded area at the head of the distributary is 30,000 hectares, and after each km it is reduced by 5,000 hectares. Out of this command, the culturable area is only 75%. The intensity of irrigation for the Rabi and Kharif seasons is 32% and 15% respectively. Design suitable channel sections for the first 3 km of this distributary, assuming the following data :*

(i) Total losses below km 3 = 0.44 cumec.

(ii) Channel losses occur @ 2 cumecs/million square metres of wetted perimeter.

(iii) Kor period for Rabi (wheat) = 4 weeks

(iv) Kor depth for Rabi = 14 cm

(v) Kor period for Kharif (Rice) = 2.5 weeks

(vi) Kor depth for Kharif = 20 cm.

(vii) Manning's n = 0.0225.

(viii) Critical velocity ratio = 0.95.

The ground levels at every 200 metres, along the line of the proposed alignment, have been obtained and are tabulated in table 4.13.

Table 4.13

Distance from head in metres	*Reduced level (G.T.S) in metres*
0	205.20
200	205.30
400	205.25
600	205.00
800	204.90
1,000	204.30
1,200	204.30
1,400	204.20
1,600	204.20
1,800	204.10
2,000	204.05
2,200	204.00
2,400	203.95
2,600	203.95
2,800	203.90
3,000	203.80

Solution. The channel is to be designed from its tail (where the losses are known) towards its head, km by km. The gross commanded areas and culturable commanded areas at various km are, first of all, worked out in Table 4.14.

Table 4.14

Below km *(1)*	*Gross commanded area in hectares (2)*	*Gross culturable area in hectares (3)*
0 (*i.e.* head)	30,000	22,500
1	25,000	18,750
2	20,000	15,000
3	15,000	11,250

Outlet discharges for the two crop seasons are determined as given below :

(*i*) *For Rabi,* $D = \frac{8.64\,B}{\Delta}$

where, $B = 4$ weeks $= 28$ days

$\Delta = 14$ cm $= 0.14$ m.

$\therefore \quad D = \frac{8.64 \times 28}{0.14} = 1728$ hectares/cumec.

(*ii*) *For Kharif,* $D = \frac{8.64\,B}{\Delta}$

where, $B = 2.5$ weeks $= 2.5 \times 7$ days $= 17.5$ days

$\Delta = 20$ cm $= 0.20$ m.

$\therefore \quad D = \frac{8.64 \times 17.5}{0.20} = 756$ hectares/cumec

Intensity of irrigation for Rabi = 32%

and Intensity of irrigation for Kharif = 15%.

If G is the gross culturable area at any point, then $0.32\,G$ is the Rabi area and $0.15G$ is the Kharif area.

Discharge reqd. for this Rabi area $= \frac{0.32\,G}{\text{Outlet factor for Rabi}} = \frac{0.32\,G}{1728} = \frac{G}{5400}$

Similarly, discharge reqd. for Kharif area $= \frac{0.15\,G}{756} = \frac{G}{\mathbf{5040}}$

Since the discharge required for Kharif crop is more than that required for Rabi crop, the outlet factor of Kharif crop becomes the controlling factor. Discharges needed at various kilometres for the given command are worked out in Table 4.15.

Table 4.15

Below km	*Gross culturable area from Table 4.14 col. (3)*	*Discharge required for Kharif crop in cumecs* $= \frac{\text{Col. (2)}}{5040}$
(1)	*(2)*	*(3)*
0	22,500	4.46
1	18,750	3.72
2	15,000	2.98
3	11,250	2.23

(*i*) **Design at km 3**

Losses below km 3 = 0.44 cumec (given)

Discharge reqd. for crop at this point (Table 4.15) = 2.23 cumecs.

Total discharge required = 2.23 + 0.44 = 2.67 cumecs

Design discharge = 10% more than required.

$= 1.1 \times 2.67 = 2.937$ cumecs, say **2.94 cumecs**

$Q = 2.94$ cumecs

$$\frac{V}{V_0} = \text{C.V.R.} = 1.0$$

$$n = 0.0225$$

Lacey's regime slope for this discharge and silt factor = 1, is approximately 22 cm per km. Let us kep the slope at 22.5 cm/km*:

Assume bed slope = 22.5 cm per km.

or $$S = \frac{0.225}{1000} = \frac{1}{4444}.$$

From Garret's diagrams [Plate 4.1 (*b*)], assuming $\frac{1}{2}$: 1 side slopes, the channel section is designed as shown in Table 4.16.

Table 4.16

Discharge	*S*	*B* m	*y* m	$A = \left(B + \frac{y}{2}\right)y$ m^2	$V = \frac{Q}{A}$ m/sec	V_0 m/sec	$\frac{V}{V_0}$	*Remarks*
2.94 cumecs	$\frac{1}{4444}$	5.0	0.98	5.38	0.55	0.53	1.04	much larger than 0.95
		4.5	1.05	5.28	0.56	0.58	0.96	O.K.

Hence, adopt $B = 4.5$ m

$y = 1.05$ m

$S = \frac{1}{4444}$ (*i.e.* 22.5 cm per km.)

These dimensions, quite nearly satisfy the bedwidth-depth relationship, given by $y = 0.5\sqrt{B}$; and hence, the assumed slope is all right and can be adopted.

(*ii*) **Design at km 2**

Outlet discharge required below km 2, from Table 4.15, col. (3) = 2.98 cumecs. Losses below km 3 = 0.44 cumec.

Losses in channel between km 3 to km 2 : For the calculation of these, the perimeter of the section at km 3 shall be taken, as the section at km 2 is not known so far.

Wetted perimeter $= B + \sqrt{5} \cdot y = 4.5 + \sqrt{5} \times 1.08 =$ **6.92 m**

Loss @ 2 cumecs/million sq. m. $= 2 \times \left[\frac{6.92 \times 1000}{10^6}\right] =$ **0.014 cumec.**

Total Losses below km 2

= Losses below km 3 + Losses between km 3 and km 2

= 0.44 + 0.014 = 0.454 cumec

Total discharge required at km 2 = 2.98 + 0.454 = 3.434 cumecs

Design discharge = 1.1 × 3.434 = 3.7874 ; say **3.79 cumecs**

Use the same slope of 22.5 cm in 1 km, *i.e.* $\frac{1}{4444}$.

* The bed slope is generally kept in multiples of 2.5 cm/km, to obtain bed levels in fraction of 0.005 m, *i.e.* the fractions up to which levels can be read on levelling staff.

Using Garret's diagrams [Plate 4.1 (*b*)], we design the channel section as shown in Table 4.17.

Table 4.17

Q	S	$\frac{B}{m}$	$\frac{y}{m}$	$A=\left(B+\frac{y}{2}\right)y\ m^2$	$\frac{Q}{A}=V$ m/sec	V_0 m/sec	$\frac{V}{V_0}$	Remarks
3.79 cumecs	$\frac{1}{4444}$	6.0	1.05	6.85	0.55	0.58	0.95	O.K.

Hence, adopt $B = 6.0$ m

$y = 1.05$ m

$S = \frac{1}{4444}$

(*iii*) **Design at km 1**

Outlet discharge required below km 1, from Table 4.15, col. (3) = 3.72 cumecs.

Losses below km 2 worked out earlier = 0.454 cumec.

Losses between km 2 and km 1 : To work out these losses, the perimeter of the section at km 2 shall be taken, as the section at km 1 is not known so far.

$\therefore$ Wetted perimeter $= B + \sqrt{5}y$

$= 6.0 + \sqrt{5} \times 1.08 = 6.0 + 2.42 =$ **8.42 m.**

Losses @ 2 cumecs/million sq. m. (*i.e.* in length of 1 km, *i.e.* 1000 m)

$$= 2 \times \left(\frac{8.42 \times 1000}{10^6}\right) = 0.017 \text{ cumec.}$$

Total Losses below km 1

$= 0.454 + 0.017 = 0.471$ cumec

Total discharge required at km 1

$= 3.72 + 0.471 = 4.191$ cumecs

Design discharge $= 1.1 \times 4.191 =$ **4.61 cumecs.**

Let us adopt a slope of 20 cm in 1 km, *i.e.* $S = \frac{1}{5000}$

Using Garret's diagrams [Plate 4.1 (*c*)], the required channel section is designed as shown in Table 4.18.

Table 4.18

Q	S	$\frac{B}{m}$	$\frac{y}{m}$	$A=\left(B+\frac{y}{2}\right)y$, m^2	$\frac{Q}{A}=V$ m/sec	V_0 m/sec	$\frac{V}{V_0}$	Remarks
4.61 cumecs	$\frac{1}{5000}$	6.0	1.2	7.92	0.582	0.615	0.947	O.K.

Hence adopt $B = 6.0$ m

$y = 1.2$ m

$S = \frac{1}{5000}$ (20 cm in 1 km.)

(*iv*) **Design at 0 km.**

Outlet discharge required at 0 km, from Table 4.15, col. (3) = 4.46 cumecs

Losses below km 1, as worked out earlier = 0.471 cumec.

Losses between km 0 to 1 : To work out these losses, the perimeter of the section at km 1 shall be taken, as the section at km 0 is not known so far.

Now, wetted perimeter

$$= B + \sqrt{5} \cdot y$$
$$= 6.0 + \sqrt{5} \times 1.2 = 6 + 2.68 = 8.68 \text{ m}$$

Losses @ 2 cumecs/million sq. m. in a length of 1 km

$$= 2\left[\frac{8.68 \times 1000}{10^6}\right] = 0.01736 = \text{say } 0.017 \text{ cumec.}$$

Total losses below km 0

$$= 0.471 + 0.017 = 0.488 \text{ cumec}$$

Total discharge required at 0 km

$$= 4.46 + 0.488 = 4.948 \text{ cumecs.}$$

Design discharge $= 1.1 \times 4.948 =$ **5.44 cumecs.**

Let us adopt a slope of 20 cm in 1 km, *i.e.* $S = \frac{1}{5000}$

Using Garret's diagrams [Plate 4.1 (*c*)], the required channel section is worked out as shown in Table 4.19.

Table 4.19

Q	S	B m	y m	$A = \left(B + \frac{y}{2}\right)y$ m^2	$V = \frac{Q}{A}$ m/sec	V_0 m/sec	$\frac{V}{V_0}$	Remarks
5.44 cumecs	$\frac{1}{5000}$	7.2	1.20	9.36	0.58	0.615	0.944	O.K.

Hence, adopt $B = 7.2$ m

$y = 1.2$ m

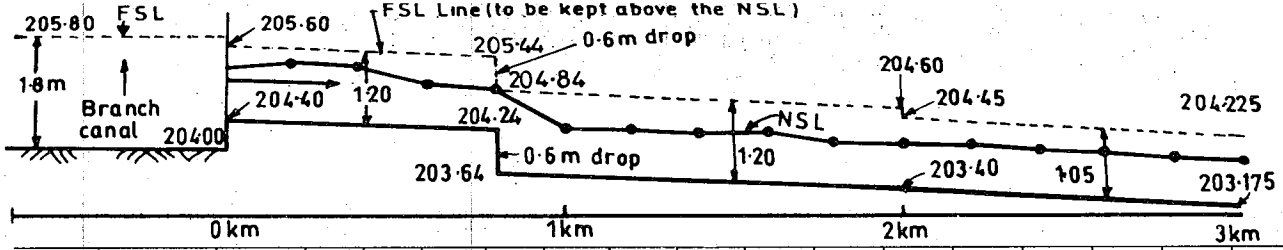
$S = \frac{1}{5000}$ (20 cm in 1 km)

All the data worked out above, has been entered at their proper places in the 'schedule of area statistics and channel dimensions' (Table 4.20). The table has been completed with the help of Canal Standards already given in Table 4.11. The L-section of the distributary is drawn, as shown in Fig. 4.30, starting from the head (*i.e.* 0 km) by keeping its FSL at head at 0.2 m below the FSL of the branch channel. The cross-sections at various km are drawn in Figs. 4.31 (*a*), (*b*), (*c*) and (*d*), by assuming $1\frac{1}{2}$: 1 slopes in filling and 1 : 1 slopes in cutting. The dowel level is kept 0.15 m higher than the bank level, and road level at 0.15 m below the bank level.

Note. The slopes of 1 : 1 shall afterwards become $\frac{1}{2}$: 1, due to silting, and that is why in design calculations, $\frac{1}{2}$ H : 1 V slopes are taken.

Table 4.20
Schedule of Area Statistics and Channel Dimensions

Below km	*Gross commanded area (hectares)*	*Culturable commanded area (hectares)*	*Controlling area to be irrigated (15%) in hectares*	*Controlling outlet discharge factor*	*Losses in reach (cumecs)*	*Total losses (cumecs)*	*Design discharge (cumecs)*	*Bed slope (cm / km)*	*Bed width (m)*	*Water depth (metres)*	*Freeboard from Table 14.11 in metres*	*Ht. of bank above ground decided after drawing L-section*	*Width of bank from Table 14.11*	*Velocity in m / sec*	$\frac{V}{V_0} = m$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*	*(14)*	*(15)*	*(16)*
0	30,000	22,500	3,775	756	0.017	0.488	5.44	20	7.2	1.2	0.6	1.00	2.25	0.58	0.944
1	25,000	18,750	2,812.5	756	0.010	0.471	4.61	20	6.0	1.2	0.5	1.00	2.0	0.582	0.947
2	20,000	15,000	2,250.0	756	0.014	0.454	3.79	22.5	6.0	1.05	0.5	0.90	2.0	0.55	0.95
3	15,000	11,250	1,687.5	756	—	0.440	2.94	22.5	4.2	1.05	0.5	0.925	2.0	0.56	0.96



Cutting depth	0.80	0.94	0.93	0.72	0.66	1.26	0.70	0.74	0.68	0.72	0.76	0.65	0.644	0.638	0.782	0.678	0.62
Designed discharge	5.44 m^3/s						4.61 m^3/s					3.79 m^3/s					2.94 m^3/s
Bed slope	20 cm/km						20 cm/km					22.5 cm/km					22.5 cm/km
Depth	1.20 m						1.20 m					1.05 m					1.05 m
Bed width	7.2 m						6.0 m					6.0 m					4.2 m
Designed FSL (m)	205.60					205.44	204.84	204.80				204.60	204.45				204.255
Ground level available (m)	205.20	205.30	205.25	205.0		204.90	204.30	204.30	204.20	204.20	204.10	204.05	204.00	203.95	203.95	203.90	203.80
Designed bed level (m)	204.40	204.36	204.32	204.28	204.24	203.64	203.60	203.56	203.52	203.48	203.44	203.40	203.355	203.310	203.265	203.220	203.175
Chainage in metres	0	200	400	600		800	1000	1200	1400	1600	1800	2000	2200	2400	2600	2800	3000
	0 km					(1 km)						(2 km)					(3 km)

Fig. 4.30. L-Section of the Distributary.

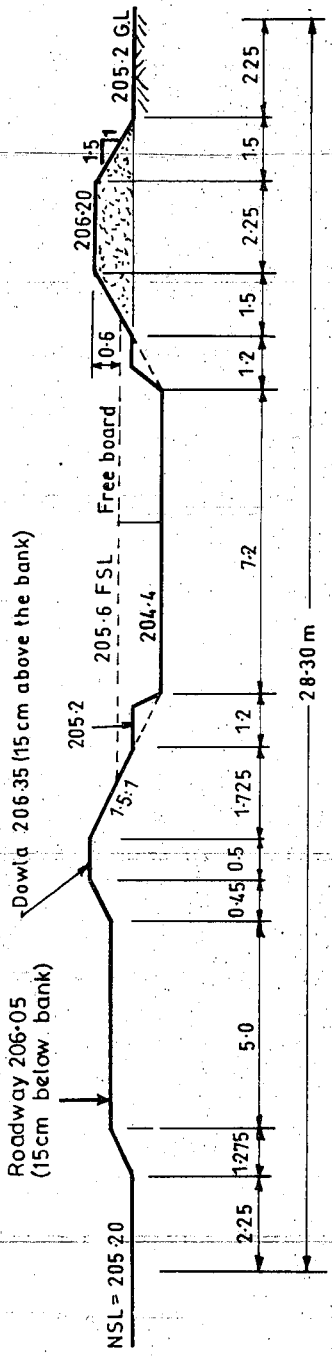


Fig. 4.31. (*a*) Section at km–0.

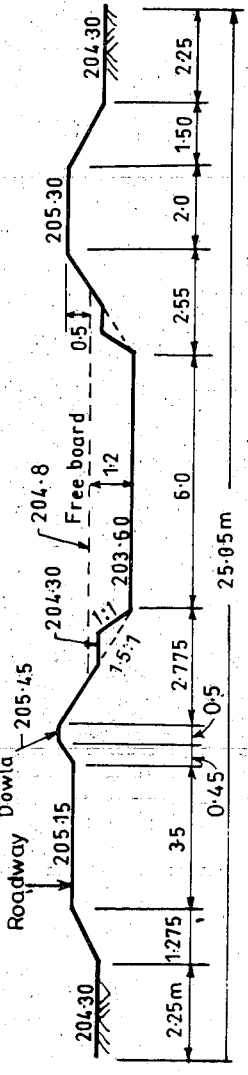


Fig. 4.31. (*b*) Section at km–1.

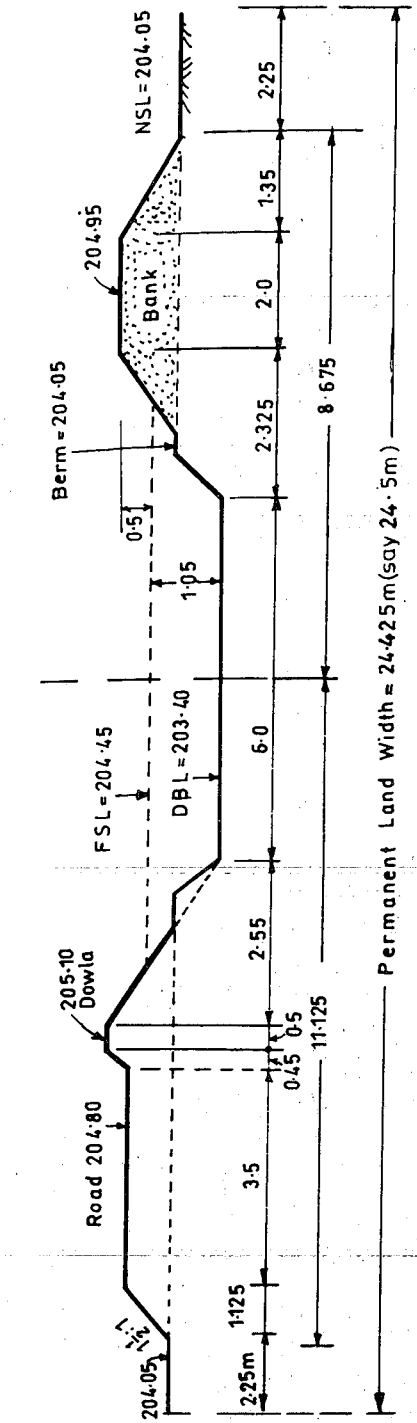


Fig. 4.31. (*c*) Section at km–2.

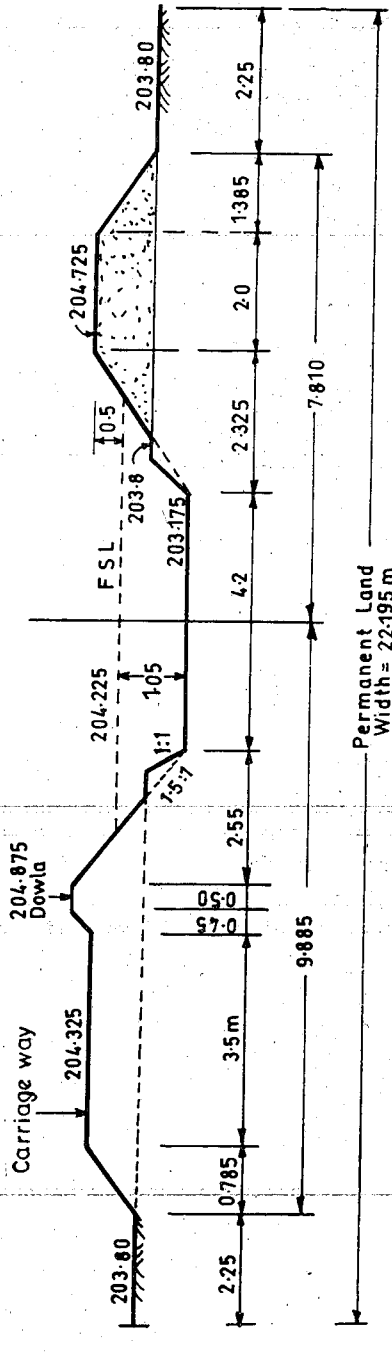


Fig. 4.31. (*d*) Section at km–3.

Example 4.17. *Design a reservoir fed canal to carry a discharge of 5 cumecs. Given Manning's N = 0.025, and the soil is of compacted silty clay with recommended maximum value of permissible velocity = 0.65 m/s.*

Solution. $Q = 5$ cumecs ; $N = 0.025$; $V_{max} = 0.65$ m/s.

From Manning's formula

$$V = \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}. \quad \text{and} \quad Q = A \cdot V$$

or $$V = \frac{Q}{A}$$

$$\therefore \quad V_{max} = \frac{Q}{A_{min}}$$

or $$0.65 = \frac{5}{A_{min}}$$

$$A_{min} = \frac{5}{0.65} \text{ m}^2 = 7.69 \text{ m}^2$$

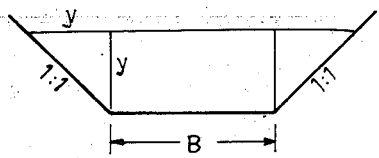


Fig. 4.32.

Since CWC's chart (Fig. 4.29) will generally not be supplied in examinations, we may solve this problem by using the relation between B and y, as given by eqn. (4.53), *i.e.*,

$$y = 0.5 \sqrt{B} \text{ (for } Q < 15 \text{ cumecs)}$$

or $$y = \tfrac{1}{2} \sqrt{B}. \qquad \text{or} \quad B = 4y^2.$$

Now, using a trapezoidal channel section with 1 H : 1 V side slopes, we have

$$A = (B + y) \cdot y. = (4y^2 + y)\, y$$

or $$4y^3 + y^2 = 7.69$$

Solve by Hit and Trial

Let $y = 1$ m L.H.S. $= 4 + 1 = 5$

Let $y = 1.2$ m L.H.S. $= 8.35$

Let $y = 1.165$ m L.H.S. $= 7.68 \approx 7.69$ (*i.e.* R.H.S.)

Hence, $y \approx 1.165$

The above calculations give only approximate value, we may hence choose depth y as anything like 1.165 m. So let us choose

$$y = 1.2 \text{ m}$$

$$A = (B + y)\, y$$

or $$7.69 = (B + 1.2)\, 1.2.$$

or $$B = \frac{7.69}{1.2} - 1.2 = \mathbf{5.21\ m.}$$

Hence, use $B = 5.21$ m, $y = 1.2$ m **Ans.**

Now, bed slope of channel can be determined as :

$$Q = \frac{1}{N} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

where $A = (5.21 + 1.2)\, 1.2 = 7.69$

$$R = \frac{A}{P} = \frac{7.69}{(B + 2\sqrt{2} \cdot y)} = \frac{7.69}{5.21 + 2\sqrt{2} \times 1.2} = 0.894 \text{ m}$$

$$\therefore \quad 5 = \frac{1}{0.025} \times 7.69 \times (0.894)^{2/3} \cdot \sqrt{S}$$

or $5 = 285.45\sqrt{S}$ or $\sqrt{S} = \frac{1}{57.08}$

or $S = \frac{1}{3260}$ **Ans.**

4.13. Maintenance of Irrigation Canals

Irrigation canals are nothing but earthwork constructions, and as such, very much susceptible to damage. They, therefore, require a lot of maintenance, upkeep, watch and ward, etc. as to ensure their continuous efficient functioning. Various problems which are posed by the irrigation canals during their use, and as such need constant attention, are : (*i*) silting of canals ; (*ii*) weed and plant growth, (*iii*) failure of weaker banks ; (*iv*) hollows created by burrowing animals, crabs, etc. in the filled up sections, causing seepage and piping ; (*v*) canal breaches due to piping, overflowing of canal, etc. Some of these factors, such as the (*i*) and (*ii*) reduce the efficiency of the canals, whereas the others may result in complete stoppage of canal supplies to the irrigated fields. All these causes of troubles and their remedial measures are discussed below :

(*i*) **Silting of Canals.** Irrigation canals normally get silted during their course of flow. Whenever the flow velocity in the channel reduces, the silt carried by the water in suspension gets deposited on the bed and sides of the canal. The silt so deposited reduces the effective canal cross-section and the carrying capacity of the channel.

In order to prevent too much of silt deposition, irrigation channels, (particularly those which are constructed in alluvial soils) must be properly designed, so as to ensure a velocity which neither causes any silting nor scouring in the channel. These design principles have already been discussed. However, even in such channels, the discharge may sometimes be less than the designed; and in that eventuality, silting may take place in the channels.

Moreover, attempts should be made to remove the silt from the water entering the canal itself. From the headworks, for this purpose, almost silt-free water is to be admitted in the canal. However, inspite of using all these measures, certain amount of silt will definitely enter the canal, and certain amount may be added to it, by some scouring from within the channel. This silt often gets deposited on the bed and sides of the canal.

The deposited silt normally deforms the shape of a channel, and reduces its carrying capacity. It must, therefore, be removed periodically by *desilting operations.*

The desilting can be carried out by actually excavating the silt either manually (when the canal is dry) or by dredgers (when the canal is either dry or running). However, if the canal functions properly and is in regime and taking its fully supply, it is not necessary to clear the silt to the theoretical cross-section. But if the canal is not functioning properly, desilting is required, but only to the extent of clearing a portion of it as to get the canal back into efficient working order.

The extent of silting that has taken place in a canal can be found from measuring the silt depth over bed bars.*

The excavated silt from the canals should be disposed of suitably depending upon its quality. Moreover, the excavated silt should not be deposited above the canal banks, as in that case, the same silt may find its way back into the desilted channel.

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*. Bed bars are the masonry walls partly extending into the canal bed and flush with it, and partly flush with the sides of the channel as shown in Fig. 13.23. For larger canals, a bed bar may consist of concrete block with its upper face flush with theoretical bed level of the canal as shown in Fig. 13.24. They serve as permanent marks of reference to indicate the correct alignment and theoretical bed levels of the channel.

(*ii*) **Weed and Plant growth.** Irrigation canals normally get infested with aquatic weed growth like bushes, hyacinth, etc. Various types of aquatic weeds and plants get grown in canals. The silted canals provide excellent base for weed growth. The sun rays also help in promoting weed growth. The weed growth hinders the flow of water, reduces the carrying capacities of channels, and thereby impairing their efficiencies. The weed growth may also sometimes impart harmful qualities to irrigation water.

Weed growths may be controlled by maintaining higher velocities in canals. Moreover, many a times, the weed growths will have to be physically removed from the canals in order to improve their efficiencies.

In case of newly constructed canals, regular inspections should be undertaken to locate any spots where the weed growth has set in. Weeds from such spots should be removed complete, so that the infestation does not spread. In the case of old canals, where aquatic weed growth is profuse, suitable mechanical or chemical methods may be employed, at as early stage as possible.

(*iii*) **Failure of Weaker Banks.** The banks of the canal should be made in full designed width (as per I.S. provisions), and should be maintained as such. Due to constant use by men and animals, the canal banks get eroded at various places. They must be repaired, and all cuts and breaches filled up with suitable soil and proper tamping.

Moreover, grass or turfing should not be scrapped while smoothening the bank surfaces, as it helps in stabilising the soil and thus preventing its erosion. However, long grasses will have to be cut as far as necessary to smoothen the surface of the bank and to avoid holes being hidden under high grasses.

Sometimes, when a canal-reach runs in filling, the banks are subjected to water pressure. In such a case, there exists every possibility of damage occurring to the banks. In such cases, the banks must be given extra strength by increasing their sections. In order to keep the cost of construction low, additional soil for increasing the bank section may be obtained by the natural silting process. For this purpose, favourable conditions are created to cause silting internally. In this system, sufficiently wider section is provided, and a part of it later gets silted, forming the additional bank as shown in Fig. 4.33. Such a method is possible only for new channels. To accelerate silting, low submergible spurs projecting from the banks into the channel may be constructed.

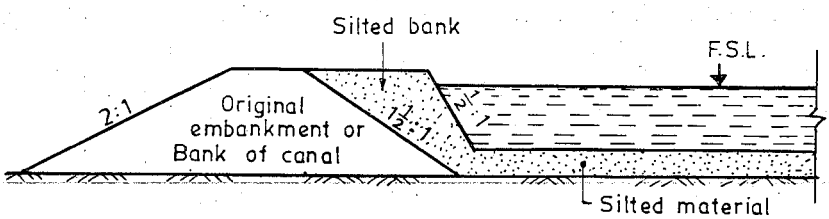


Fig. 4.33. Internal silting method for widening canal banks in canals constructed in filling.

(*iv*) **Canal Breaches.** Canal breach is, infact, an opening or a gap developed in the canal bank due to erosion of some portion of the bank. When once water starts flowing out of such a breach, it starts becoming deeper & wider, unless remedial measures are

taken immediately to plug the breach. The breaches in canal banks may be caused due to various reasons, such as :

(*a*) breach due to faulty design or construction of the canal banks ;

(*b*) breach due to overflow of the canal ;

(*c*) breach due to seepage or piping ;

(*d*) breach due to intentional cuts made by cultivators.

These reasons are briefly summarised below :

(*a*) **Breach due to faulty design or construction of the banks.** If the width of a canal bank is not kept sufficient, or if its height is not as per the correct design requirements, the bank may fail to hold the water properly. Similarly, if the soil material used at site is poor and not up to the specifications or if the rolling and watering, etc. has not been properly done at the time of construction, the banks may not be so strong as were expected to be, and hence fail to hold the canal water without being damaged. This continuous recurring damage to the bank may ultimately lead to a full-fledged cut and breach within the bank length.

(*b*) **Breach due to overflow of canal.** Sometimes, the discharge in the canal may exceed its design capacity, and water may overtop the canal bank(s). The overflow will definitely damage the bank(s) considerably.

(*c*) **Breach due to piping.** Sometimes, the banks may not be overtopped, but the water depth may go above the full supply depth due to increased discharge or reduced silted canal section. In such a case, the hydraulic gradient line within the 'filled' canal bank section will also rise (as shown in Fig. 4.34). And if the provided cover over the hydraulic gradient line is not sufficient, it may pass out of the banks, thus dislodging the soil particles from the outer slopes of the banks. It will consequently weaken and erode the banks, which may ultimately lead to a complete breach within the bank, and water may rush out of the canal.

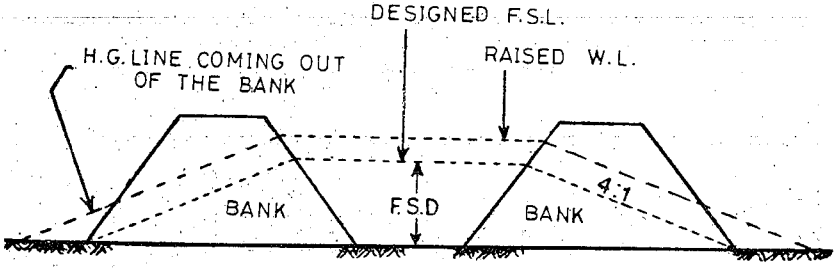


Fig. 4.34.

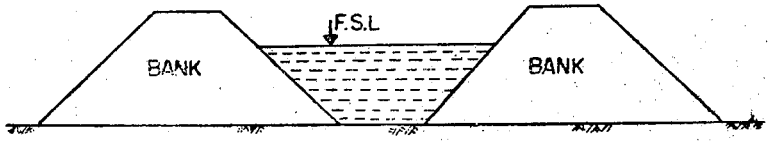
Sometimes, the water may start leaking through the holes created within the body of the bank, by insects or burrowing animals. The leaking water will go on removing the sand particles alongwith ; and if went unchecked, the holes may go on increasing in size, and ultimately resulting in a rush of water through the canal bank, leading to sinking of the bank soil, and formation of a full fledged breach. Normally, however, these holes are smaller in size and do not pose a serious problem, if timely inspection and remedial plugging is done. As a preventive measure, sometimes, a sand core may be provided within the bank embankment section, which will settle and fill the holes. In such a case, of course, the banks will settle down, but at least breaches will be avoided.

(*d*) **Breach due to intentional cuts made by cultivators.** Sometimes, when the area gets flooded due to excessive rain or poor drainage, the cultivators cut the canal bank in order to pass the drainage water to the other side. This sometimes proves rather detrimental to their expectations, as the canal supplies may also enter the low lying area, thus further aggravating the water congestion in the area.

Sometimes, similarly, the cultivators cut the banks to obtain illegal water supplies from the canal. These small cuts may widen and take the shape of big breaches in a short time, consequently damaging the land and crops.

Closure of breaches. In case of *small minors and distributaries,* a breach may be closed by dumping huge quantities of earth instantaneously from both sides of the gap. As the water of the breach spreads on the adjoining land, there is usually no outside nearby place to borrow earth for closing the breach. The earth has, therefore, to be collected either by cutting the outer slope of the existing bank, or from the spoil banks (if existing), or berms of the canal. It is essential to store huge quantities of earth on both sides of the gap before closing the breach. Moreover, the closing process should be started from both sides of the breach, at the same time, as pointed out earlier.

In case of *bigger canals,* such as a major distributary or a branch canal, the above procedure cannot be adopted, as the huge discharge from such a canal may completely wash off the dumped soil. In such a case, it is, therefore, necessary to, first of all, reduce the flow through the breach. This should be done by driving a double line of stakes or



SECTION

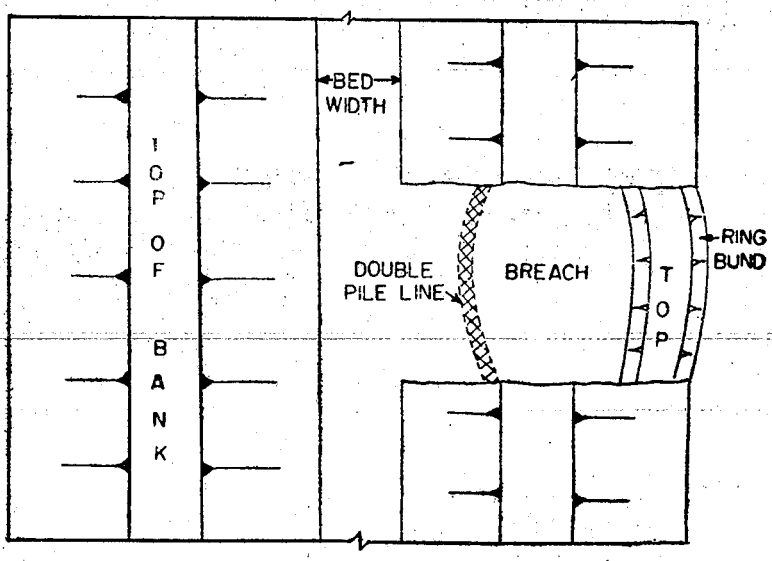


Fig. 4.35. Closure of a breach in a canal bank.

wooden piles (ballies) in the opening of the breach, as shown in Fig. 4.35. The space between the pile lines is filled with planks or bushes, etc. The filler material can be secured by placing sand bags on the top. If the breach is very wide, another line of defence may be provided.

This will reduce the outflow from the breach. No dumping of earth in the breach should progress before the flow through the breach has been arrested to some extent in this fashion. Meanwhile earth should be piled up on the bank on both sides of the breach.

This deposited earth should then be dumped instantaneously from both sides to form a *ring bund* on the outer side of the breach as shown. The opening is then properly filled with suitable earth in layers, each layer being properly compacted. All jungle from the ring bund site should be removed before earth work progresses. Moreover, the earth should be freed from grasses or bushes, etc., before dumping.

4.14. Modern Simplified Equations for Optimal Designs of Alluvial Canals

While discussing the design procedure by Kennedy's theory, we have illustrated that the alluvial canals are normally designed to satisfy known values of discharge (Q), rugosity coefficient (n), and silt grade (m). The value of longitudinal slope (S) is also usually predecided on the basis of ground considerations. Thus, the design is usually carried out for known values of Q, n, m & S to satisfy Kutter's equation as well as Kennedy's equation, both. Such a design will give us dimensions of a channel which will be hydraulically adequate, (indicated by Kutter's Eqn.) and will also be non-silting-non scouring (indicated by Kennedy's eqn.). *But infact, such a channel may not behave satisfactorily from considerations of silt transportation.* This is verified by experience in the field. Evidently, some guidance for fixing a suitable bed width - depth ratio $\left(\frac{b}{y}\ ratio\right)$ is required. These ratios were fixed in various areas on the basis of experience, or by empirical equations and charts like the ones given in eqn. (4.62) and Fig. 4.29. The resulting solution obtained for channel dimensions (with given values of Q, n, m *and* S) is finally compared with the recommended values of $\frac{b}{y}$ from Fig. 4.29, or Eq. (4.62). If the two values differ significantly, suitable modification in the bed slope (S) would become necessary. Since the slope is decided on the basis of ground profile, it will limit the range of slope. Within this range of slope, one can obtain different combinations of b and y satisfying Kennedy's and Kutter's equations and the curve in Fig. 4.29. All such computations are cumbersome and time consuming. To avoid time consumption, Garret's diagrams are used, which provide graphical solution to Kennedy's and Kutter's equations. However, this method involves interpolation errors. Considering these drawbacks of the age old design method, a need was being felt to develop explicit type of equations, which will satisfy both Kennedy's and Kutter's equations, without involving trial and interpolation errors.

Some modern equations have, therefore been developed to overcome the above shortcomings of the old age method, and are given below.

Considering a trapezoidal channel section with $\frac{1}{2}H:1V$ side slope, we have from Kennedy's Eqn.

$$V = 0.55\, m \cdot y^{0.64} \qquad \ldots(i)$$

Also from continuity eqn., we have

$$Q = A \cdot V = y\left(b + \frac{1}{2} \cdot y\right) \cdot V \qquad ...(ii)$$

Eliminating V between (i) and (ii), we get

$$Q = y(b + 0.5y)\, 0.55 \cdot m \cdot y^{0.64}$$

$$= 0.55m \cdot y^{1.64}(b + 0.5y)$$

$$= 0.55 \cdot m \cdot y^{1.64} \cdot y\left(\frac{b}{y} + 0.5\right)$$

or $$Q = 0.55\, m\left(\frac{b}{y} + 0.5\right) y^{2.64}$$

Substituting $\frac{b}{y} = r$, we get

$$Q = 0.55\, m \cdot (r + 0.5)\, y^{2.64}$$

or $$y = \left[\frac{Q}{0.55 \cdot m\,(r + 0.5)}\right]^{\frac{1}{2.64}}$$

or $$y = \left[\frac{1.818\, Q}{(r + 0.5)\, m}\right]^{0.3788} \qquad ...(4.63)$$

where $$\frac{b}{y} = r \qquad ...(4.64)$$

One can compute both b and y from Eq. (4.63) & (4.64), if a suitable explicit equation written symbolically as :

$$r = f(Q, n, m, S) \qquad ...(iii)$$

is available, which satisfies both Kennedy's and Kutter's equations. A large number of canal designs can be obtained by varying the parameters Q, n, m and S in their practical ranges and a suitable correlation can be established between r, Q, n, m and S.

By varying the parameters r, Q, n, m and S, in their practical ranges, 1296 number of canal designs were obtained by satisfying both the Kennedy's and Kutter's equations. Analysing all the resulting solutions obtained earlier, it is found that a unique relation could be established between r, Q, n, m and S. By a method of curve fitting, the following empirical formula has been obtained :

$$\boxed{r = \left[\frac{1.607\, S^{1.63} \cdot Q^{0.033}}{n^{3.26} \cdot m^{3.293}} - 0.258\right]^{-0.915}} \qquad ...(4.65)$$

For known values of Q, n, m and S, value of r can be computed from the above eqn. With this value of r, the value of y can be calculated from Eq. (4.63), and then the value of b can be calculated by Eqn. (4.64). The resulting solution may finally be compared with the curve of Fig. 4.29. If the matching differs widely, a modification in S value would be necessary and may be done in the following manner.

The curve of Fig. 4.29 can be represented by an equation :

$$r = \left[15 + 6.44\, Q\right]^{0.382} \qquad ...(4.66)$$

Eq. (4.66) can be considered as the optimality condition for the value of r. On combining Eqns. (4.65) and (4.66), we get

$$S = \frac{n^2 \cdot m^{2.02}}{1.338 \cdot Q^{0.02}} \left[0.258 + (15 + 6.44\, Q)^{-0.417}\right]^{0.6135} \quad ...(4.67)$$

Eq. (4.67) will give optimal value of S, which will satisfy the curve of Fig. 4.29. However, if a particular value of S is to be maintained, the design may be carried out by relaxing the requirement of r.

Design Steps

For given values of Q, n and m the proposed design steps are :

(*i*) Compute the optimal value of S-from Eqn. (4.67).

(*ii*) Find the required value of r from Eq. (4.66).

(*iii*) Find y from Eq. (4.63).

(*iv*) Find b from Eq. (4.64).

If the slope is to be maintained at a given value, then the computation steps will be as follows :

(*i*) Find r-from Eq. (4.65)

(*ii*) Find y-from Eq. (4.63)

(*iii*) Find b-from Eq. (4.64).

Example 4.18. *Design an irrigation channel to carry 50 cumecs of Discharge. The channel is to be laid at a slope of 1 in 4000. The critical velocity ratio (m) for the soil is 1.1. use Kutter's rugosity coefficient (n) as 0.023. Solve by using the modern equations.*

Note. *This problem is the same as was there in Example 4.6, which was solved by old methods.*

Solution. $Q = 50\ \text{m}^3/\text{s}$; $S = \dfrac{1}{4000}$

$m = 1.1$; $n = 0.023$

(*i*) *Find r by using Eq. (4.65) as :*

$$r = \left[\frac{1.607 \cdot S^{1.63} \cdot Q^{0.033}}{n^{3.26} \cdot m^{3.293}} - 0.258\right]^{-0.915}$$

Substituting the given values of S, Q, n and m, we get

$$r = \left[\frac{1.607 \cdot \left(\dfrac{1}{4000}\right)^{1.63} \cdot (50)^{0.033}}{(0.023)^{3.26} \cdot (1.1)^{3.293}} - 0.258\right]^{-0.915}$$

$$= \left[\frac{1.607 \times 1.3447 \times 10^{-6} \times 1.1378}{4.5628 \times 10^{-6} \times 1.36869} - 0.258\right]^{-0.915}$$

$$= (0.3937 - 0.258)^{-0.915} = (0.1357)^{-0.915}$$

$$= \frac{1}{(0.1357)^{0.915}} = 6.219$$

(*ii*) *Now find y, by using Eq. (4.63) as :*

$$y = \left[\frac{1.818\,Q}{(r+0.5)\,m}\right]^{0.3788} = \left[\frac{1.818\times 50}{(6.219+0.5)\,1.1}\right]^{0.3788} = 2.587 \text{ m ; say } 2.59 \text{ m.}$$

(*iii*) *Find b from Eq. (4.64) as :*

$$\frac{b}{y} = r \qquad \therefore \quad b = yr = 2.59 \times 6.219 = 16.1 \text{ m}$$

The dimensions of the designed channel will hence be :

$$\left.\begin{aligned} B &= \text{bed width } (b) = 16.1 \text{ m} \\ y &= \text{water depth} = 2.59 \text{ m} \end{aligned}\right] \quad \textbf{Ans.}$$

Note 1. Compare this design with the design done in Example 4.6, where we obtained $B = 14.14$ m and $y = 2.7$ m

Note 2. These modern equations can be easily used to design canals, which would not only satisfy Kutter's & Kennedy's Eqns. but also provide desired $\frac{B}{y}$ ratios.

Note 3. Such equations can be easily used in digital computer programmings for design of irrigation canals.

Example 4.19. *Design an irrigation channel to carry 40 cumecs of discharge with an optimum value of Bedwidth/Depth ratio. The value of rugosily coefficient n = 0.023, and critical velocity ratio, m = 1.*

Solution. (*i*) Use Eq. (4.67) to determine optimal value of bed slope S, as

$$S = \frac{n^2\, m^{2.02}}{1.338\, Q^{0.02}}\left[0.258 + (15 + 6.44\,Q)^{-0.417}\right]^{0.6135}$$

where $n = 0.023$

$m = 1$

Substituting values, we get

$$S = \frac{(0.023)^2\,(1)^{2.02}}{1.338\,(40)^{0.02}}\left[0.258 + (15 + 6.44\times 40)^{-0.417}\right]^{0.6135}$$

$$= 3.6725 \times 10^{-4}\left[0.3544\right]^{0.6135} = \mathbf{1.944 \times 10^{-4}}$$

(*ii*) Find r from Eq. (4.66)

$$r = \left[15 + 6.44\,Q\right]^{0.382} = \left[15 + 6.44\times 40\right]^{0.382} = \mathbf{8.519}$$

(*iii*) Find y from Eq. (4.63) as :

$$y = \left[\frac{1.818\,Q}{(r+0.5)\,m}\right]^{0.3788}$$

$$= \left[\frac{1.818\times 40}{(8.519+0.5)\,1}\right]^{0.3788} = 2.20 \text{ m}$$

(*iv*) Find b from Eq. (4.64) as :

$$\frac{b}{y} = r$$

or $$b = yr = 2.20 \times 8{,}519 = 18.78 \text{ m}$$

The designed dimensions of the channel are :

$$\left.\begin{array}{l}\text{bed width, } b = 18.78 \text{ m} \\ \text{water depth, } y = 2.20 \text{ m} \\ \text{Bed slope, } S = 1.944 \times 10^{-4} = \dfrac{1}{5144}\end{array}\right] \textbf{Ans.}$$

Please compare the results which were obtained for this data in Example 4.7 as $b = 8.35$ m, $y = 3.34$ m, $S = 1$ in 3700].

Example 4.20. *Make an alternative design for the canal at km 3 in Example 4.16 by using Modern Equations in place of using Gerret's Diagrams as were used in Table 4.16.*

Solution. The data relating to canal design at km 3, was :

$$Q = 2.94 \text{ m}^3/\text{s} ; \qquad n = 0.0225$$

$$S = \frac{1}{4444} = 0.225 \times 10^{-3}$$

$$\text{Side slopes} = \frac{1}{2} : 1 ; \qquad \text{CVR} = m = 1$$

Now, instead of designing this canal section by Garret's diagrams, we can design it by using modern equations of optimal canal design, as follows :

(*i*) Find r from Eq. (4.65) as :

$$r = \left[\frac{1.607\,(0.225 \times 10^{-3})^{1.63} \cdot (2.94)^{0.033}}{(0.0225)^{3.26} \cdot (1)^{3.293}} - 0.258\right]^{-0.915} = 4.659$$

(*ii*) Find y from Eq. (4.63) as :

$$y = \left[\frac{1.818 \times 2.94}{(4.659 + 0.5)\,1}\right]^{0.3788} = \mathbf{1.01}$$

(*iii*) Find b from Eq. 4.64

$$b = y \cdot r = 1.013 \times 4.659 = \mathbf{4.72 \text{ m}}$$

Hence, adopt $B = 4.72$ m, $y = 1.01$ m. Compare these figures with the values worked out in Table 4.16 by Garret's diagrams as : $B = 4.5$ m, $y = 1.05$ m.

Note. In this manner, the use of Garret's diagrams can be completely avoided.

PROBLEMS

1. What is sediment ? How is sediment transported in streams ? What are dunes and antinodes ? Write an equation giving the relation between total sediment transport and streamflow. Explain how would you estimate the life of a reservoir.

2. What is the difference between suspended load and bed load ?

State the better empirical bed load formulae. Give briefly the theory of distribution and transportation of suspended matter. What is Einstein's approach ?

How is suspended sediment measured and expressed ?

3. (*a*) Explain the following terms connected with sediment transport phenomenon:

(*i*) Suspended load (*ii*) Bed load

(*iii*) Regime channel (*iv*) Lacey's silt factor

(*v*) Threshold motion of the sediment.

(*b*) In the design of a trapezoidal canal, the following dimensions are obtained :

side slopes = 2 vertical to 1 horizontal
bed width = 21 m
depth = 1.5 m
bed slope = 2.25×10^{-3}

Manning's friction factor for the canal was $n = 0.022$. Using Kennedy's equation on silt theory, given by $V_0 = 0.55, D^{0.64}$ with usual notations, find whether the chosen canal section is satisfactory.

4. (*a*) Discuss the mechanics involved in Sediment transport.

(*b*) Explain in details, the Shield's method for design of channels with pitched slopes.

(*c*) An irrigation channel is to be constructed in coarse alluvium gravel with 4 cm size. The channel has to carry 5 cumecs of discharge and the longitudinal slope is 0.04. The banks of the channel will be pitched against scouring. Find the minimum width of the channel.

[**Hint :** Follow example 4.1.]

5. (*a*) Derive an expression for the average tractive force per unit of wetted area that is generated in a trapezoidal channel section of given R and S. How does this shear stress distribution at banks differs from that at bed ?

(*b*) Also derive an expression to prove that the shear stress required to move a grain on the bank of a channel is lesser than the shear stress required to move the grain on the channel bed.

6. (*a*) Discuss critically the statement "the banks of an unlined canal are more susceptible to erosion than its bed, and hence the stability of the banks and not of its bed is the governing factor in unlined canal designs".

(*b*) A canal is to be designed to carry a discharge of 600 cumecs. The bed slope is kept as 1 in 1600. The soil is coarse alluvium having a grain size of 5 cm. Assuming the canal to be unlined with unprotected banks and of a trapezoidal section, determine a suitable section for the canal ; ϕ for the soil may be taken as 37°.

[**Hint :** Follow example 4.3]

7. What is meant by "regimen of a river" ?

Compare briefly the silt theories of Kennedy and Lacey.

Design a regime channel to carry a discharge of 50 cumecs. Assume silt factor as 1.0.

8. When do you classify the channel as having attained regime condition ?

Describe briefly the observations of Lacey on the regime of river.

Design a channel using Lacey's theory to carry a discharge of 100 cumecs. Assume silt factor as 1.

9. Describe briefly the two recognised silt theories. Explain how one theory is an improvement over the other.

What are defects in both these theories.

10. What is meant by 'regime' ? Differentiate between regime in natural rivers and in artificial channels.

Design an irrigation channel section for the following data :

Discharge = 40 cumecs
Silt factor = 1.0
Side slopes = $\frac{1}{2}$: 1 ;

Determine the longitudinal slope also.

11. Discuss the salient features of Kennedy's theory for the design of earth channels based on the critical velocity concept, and mention its limitations.

Also, design an earth canal section to carry 50 cumecs discharge at a slope of 0.25 m/km, having been given that $N = 0.0225$, and $m = 1.00$, where the symbols have their usual meaning.

12. Explain what is meant by 'unlimited incoherent alluvium' in the context of Lacey's theory and discuss the concept of regime embodied in this theory for the design of earth channels in alluvium.

Design an irrigation channel to carry 40 cumecs at a slope of 1 in 5000 with $N = 0.0225$ (Manning's) and Kennedy's $m = 0.9$.

13. (*a*) Differentiate between 'Initial regime' and 'Final regime'.

(*b*) Design an irrigation channel section for the following data :

Discharge = 30 cumecs ; Silt factor = 1.0 ; Side slopes = $\frac{1}{2}$: 1.

Draw the complete channel cross-section assuming it to be in part cutting and part filling.

14. (*a*) Discuss briefly the problems that arise and the methods which are adopted while designing the sections for irrigation canals in India.

(*b*) An area of 40,000 ha. has to be irrigated by a canal for growing wheat ; water requirement for which is 10 cm per month. Design and draw a suitable canal section with the data given below :

Mean slope of the ground = 1 in 3400,

Manning's roughness coefficient = $N = 0.025$,

Side slopes = 1.1.

Use Kennedy's formula. Try a depth equal to 2 m.

$$\left(\textbf{Hint:}\quad Q = \frac{40.000 \times 10^4 \times 0.1}{30 \times 24 \times 60 \times 60}\ \text{cumecs}\right)$$

15. Write detailed notes on any two of the following :

(*i*) Importance of sediment transport in designing earthen irrigation canals.

(*ii*) Kennedy's and Lacey's silt theories for designing irrigation canals in India.

(*iii*) Comparison of Kennedy's and Lacey's silt theories, and further improvements over Lacey's theory.

(*iv*) Use of Garret's Diagrams for designing irrigation canals.

(*v*) Suspended load and its measurement.

(*vi*) Bed load and its measurement.

(*vii*) Popular Methods of designing irrigation canals in western countries.

16. (*a*) What is meant by 'Suspended load' in an irrigation canal ?

(*d*) Derive an expression for measuring the suspended load concentration at any distance y from the bed of the channel, provided the concentration at a known distance is already known.

[**Hint :** Derive equation 4.34 and 4.40]

17. (*a*) Define bed load, and discuss Meyer-Peter's formulae for determining bed load in irrigation canal.

(*b*) Design a channel which has to carry 30 cumecs of discharge with a bed load concentration of 40 p.p.m. by weight. The average grain diameter of the bed material may be taken as 0.35 mm. Use Lacey's regime perimeter and Meyer-Peter's formulas.

[**Hint :** Follow example 4.10]

18. Derive Einestein's formula for bed load transport in a form which can help in deducing that the 'silt carrying capacity of a river channel increases during floods'. What other interesting conclusions can be drawn from this formula ?

[**Hint :** Derive Equation 4.65]

19. (*a*) Draw a typical canal cross-section which is partly constructed in cutting and partly in filling. Discuss briefly its various components, such as : side slopes, berms, banks, service road, dowla, spoil banks, etc.

(*b*) What is meant by 'Balancing depth and how is it determined' ?

(*c*) The following data refer to an irrigation canal

Bed width	= 1.0 m
Side slopes	= 2 : 1 (in filling) and 1 : 1 (in cutting)

Top width of embankment on either side of canal = 3 m

Full supply depth	= 5 m.
Free board	= 1 m.

Determine the balancing depth. Draw a neat cross-section of the canal illustrating the various dimensions and the levels.

(Take ground level as 100.00 m)

20. Write short notes on :

(*i*) Free board in canals. (*ii*) Berms and counter berms.

(*iii*) Requirements of permanent land widths for constructing canals.

(*iv*) Spoil Banks. (*v*) Borrow pits.

(*vi*) Balancing depth.

(*vii*) Fixing bed width ratio for an irrigation canal.

(*viii*) Canal standards. (*ix*) Schedule of area statistics.

21. Describe briefly as to what data you will collect and how you will proceed for fixing the L-section of an irrigation canal. Also discuss step by step, the procedure that you will adopt for designing the entire length of the channel section, if the culturable commanded area and the details of the crops along the canal are given.

5

Lining of Irrigation Canals and Economics of Lining

5.1. General

Next to China, India has the largest area under irrigation. The irrigated area is constantly and continuously being increased, so as to ensure assured irrigation and to avoid crop failures due to famines and non-availability of water. To secure the benefits of irrigated land, a tremendous amount of capital has been invested in this country in the irrigation projects. So much so, that an expenditure of about Rs. 1,15,000 crores has been incurred in our country on major and medium irrigation projects since independence and up to the end of ninth five-year plan (1997–2002). This data confirms the fact that irrigation water is a costly commodity, and as such, there should be no wastage during its carriage from the reservoirs to the fields. Most of the canals, constructed in India to carry irrigation water, are unlined, and hence, a large part of the costly irrigation water is lost in percolation and absorption as seepage loss. No doubt, there are regions where the soil is such that seepage losses are very small, and there is no justification for lining them, but at the same time, it is also true that there are areas where 25 to 50% of the water is lost in seepage. This is a very serious loss and proportionately reduces the irrigation potential of the supplied water. Such seepage loss of the costly irrigation water must, therefore, be minimised. The seepage can be avoided or minimised by *lining the irrigation canals.*

By lining the canal, we mean that the earthen surface of the channel is lined with a stable (inerodible) lining surface, such as concrete, tiles, asphalt, etc. Depending upon the type of lining adopted, the seepage losses can be reduced to 2%–5% of their original values by lining the canals. Apart from reduction in seepage losses, there are various other advantages of lining the canals. All these advantages are described below in details :

5.2. Advantages of Lining

(1) **Seepage Control.** It has been emphasised above, that the seepage losses are considerably reduced if the channels are lined. A lined canal costs about 2 to $2\frac{1}{2}$ times as much as an unlined canal, but where seepage is heavy, the saving of costly irrigation water may itself be sufficient to fully justify the capital expenditure on lining. It should also be kept in mind that heavy seepage losses in canals would necessitate the construction of larger reservoirs and bigger dams. Prevention of seepage by lining would, therefore, reduce their impounding capacity, and hence, lower the construction costs of these works. The idea of the extent to which seepage takes place in different kinds of channels can be obtained from Table 5.1.

Table 5.1. Seepage Losses in Channels

S. No.	*Type of lining*	*Initial rate of seepage in cumecs per million sq. m. of wetted area before lining*	*Stabilised rate of seepage in cumecs per million sq. m. of wetted area after lining*
1.	Unlined channels.	7.4	3.4
2.	30 cm × 15 cm × 5 cm tiles using cement mortar 1 : 3 (1 cement ; 3 sand in between two layers.	0.17	0.009
3.	10 cm thick cement concrete 1 : 3 ; 6 (1 cement : 3 sand : 6 brick ballast.	0.13	0.007
4.	10 cm thick cement lime concrete 1 : 5 : 12 : 24 (1 cement : 5 lime : 12 surkhi : 24 brick ballast).	0.40	0.13

(2) **Prevention of Water-logging.** Uncontrolled seepage through unlined canals, often raises the water-table in the surrounding fields up to or near to the ground level, as to bring the crop (plant) roots within the capillary fringe. This, in turn, brings up the alkali salts near to the ground surface, which soon renders the land unfit for cultivation. Such a land is usually called *thur*, and the phenomenon of rise of water table is known as water-logging.* It is well-known that large tracts of lands in Punjab (India and Pakistan) have been rendered uncultivable in this manner. Thus, the very canal, built to increase food production, have within a few years, caused the ruin of a large portion of agriculturable land. Lining of canals prevents seepage and, thus, protects cultivable land. Combined with land drainage schemes, lining helps to reclaim water-logged areas.

(3) **Increase in Channel Capacity.** The capacity of a chosen canal section can be considerably increased by lining it. The reason is very simple. The lining presents a smooth surface and, therefore, causes less resistance to the flow of water. The water, therefore, flows faster, and hence, more of it is carried per second than that in an unlined canal. Flow of water in unlined canals, is often, further retarded by the vegetation on the sides and bottom. And since capacity is a function of velocity, the higher the velocity, the greater is the capacity of the channel.

We know that Kutter's formula for channel velocity is

$$V = \left[\frac{23 + \dfrac{0.00155}{S} + \dfrac{1}{n}}{\left(23 + \dfrac{0.00155}{S}\right)\dfrac{n}{\sqrt{R}} + 1} \right] \sqrt{RS}$$

where n is the coefficient of roughness. The velocity, therefore, varies inversely with n. For unlined earth channels in good condition, the value of n is about 0.0225 against 0.012 for cement mortar, 0.015 for concrete, and 0.018 for brick lined channels. Table 5.2 gives commonly used n values for different types of linings.

* For details, please see Chapter 6.

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Table 5.2. Values of Mannings (n) for Lined Channels

Type of Lining	*Values of n for straight alignment**
Cast insitu concrete trowel finish	0.015—0.018**
Cement plastered masonry	0.012—0.015
P.C.C. slabs or tiles	0.018—0.020
Brick lining	0.018—0.020
Split Boulder lining	0.020—0.025
Round Boulder lining	0.025—0.035

Lining, therefore, increases channel capacity and consequently reduces the required channel section. Hence, for a new designed project, a lined channel will require lesser dimensions and hence, lesser earth work. The consequent saving in earth work handling (*i.e.* excavation and filling) and acquisition of land, thus, become possible by canal lining.

(4) **Increase in Commanded Area.** A lined canal can be designed not only smaller in cross-section but also shorter in length. The steeper gradients can be provided because higher velocities are permissible (as the material is less erodible) and a shorter alignment can, therefore, be selected. On the other hand, flatter slopes can be provided without silting on a lined channel compared to these on an unlined channel. It can, therefore, help to bring high areas under command.

(5) **Reduction in Maintenance Costs.** The up-keep of unlined canals involve huge recurring expenditure, generally charged under the head of account : *A.R.* and *M.O.* of the canal system (*i.e.* Annual repair and maintenance of the canal system). This expenditure may be required on :

(*i*) periodical removal of silt deposited on the bed and sides of the canal section ;

(*ii*) minor repairs like plugging of cracks, cuts and uneven settlements of banks ; and

(*iii*) removal of weeds and water plants.

The provision of lining reduces these charges considerably, as the cost of upkeep of a lined canal is comparatively negligible.

The above three heads are discussed below in details :

(*i*) *Removal of silt.* A lined canal is not susceptible to erosion. It is usually designed to carry the sediment load likely to enter at the canal headworks. Moreover, on account of the high velocities in lined channels, the sand blown into it during sand storms, which may occur during summer in areas like Rajasthan deserts, etc. is readily carried away. This eliminates or considerably reduces the annual expenditure required on unlined channels for desilting.

(*ii*) *Minor repairs.* Periodical plugging of holes burrowed by rats, insects, etc., is constantly required in unlined channels, failing which, breaches of channel embankments may occur. The provision of adequate lining, reduces the danger of these breaches, and lesser vigilance is required.

* For canals with non-straight alignments, loss of head by resistance to flow shall increase, and hence the given n values may be slightly increased.

** Lower values may be attained in canals with relatively higher discharges and in absolutely straight reaches.

(*iii*) *Removal of weeds.* Huge money is spent in removing weeds and water plants like hyaocinth, etc., from the canals. Lining eliminates or reduces this expenditure considerably, as the plants flow down the canal due to higher flow velocites in lined canals.

(6) **Elimination of Flood Dangers.** An unlined canal founded on weaker foundations is always in danger, and a breach may occur at any time. Instances have occurred where small breaches in unlined canals resulted in washing away of considerable length of embankment—leading to flooding of certain areas and causing scarcity of irrigation water in others, as the canal was out of service at a critical time for crops. A strong concrete lining removes all such dangers.

FINANCIAL JUSTIFICATION AND ECONOMICS OF CANAL LINING

In certain cases, the lining of a channel may be required from purely technical considerations. For example, a canal constructed on a high fill or a canal founded partly on rock and partly on permeable strata, may be unsafe, unless it is lined. Sometimes a hard lined surface may be required to withstand the high flow velocities, as in power channels. Apart from these special circumstances, the engineer is required to produce a good economic justification for the capital outlay that is likely to be invested on lining. In considering the economy of canal lining, it is necessary to evaluate the tangible (which can be measured in terms of money) and additional benefits, and then to compare these with the cost of lining. *Benefit cost ratio* can, therefore, be worked out, so as to justify the necessity of lining.

Mathematically speaking, expenditure on a project is justified if the resultant annual benefits exceed the annual costs (including interest on the capital expenditure) *i.e. Benefit cost ratio is more than one.* The justification for lining the existing channels is different from that of constructing new lined channels in a new project. It is because of the fact that a large number of additional advantages ; such as lesser earth-work-handling, lesser land acquisition, lesser impounding reservoir capacities, etc., are obtained in a new project, by adopting lining for new canals.

5.3. Justification for Lining the Existing Canals

(*i*) **Annual Benefits.** Irrigation water is sold to the cultivators at a certain rate. Let this rate be rupees R_1 per cumec. If m cumecs of water is saved by lining the canal, annually, then the money saved by lining $= mR_1$ rupees.

Lining will also reduce maintenance cost. The average cost of annual upkeep of unlined channel can be worked out from previous records. Let it be Rs. R_2. If p is the percentage fraction of the saving achieved in maintenance cost by lining the canal, then the amount saved $= p \cdot R_2$ rupees.

$$\therefore \text{The total annual benefits} = mR_1 + p \cdot R_2 \qquad ...(5.1)$$

(The value of p is generally taken as 0.4)

(*ii*) **Annual Costs.** If the capital expenditure required on lining is C rupees, and the lining has a life of say Y years, then the annual depreciation charges will be $\frac{C}{Y}$ rupees. If r percent is the rate of annual simple interest, then a locked up capital of C rupees would earn, annually $C\left(\frac{r}{100}\right)$ rupees as interest charges, and since the capital value of

the asset decreases from C to zero in Y years, the average annual interest cost may be taken as $\frac{C}{2}\left(\frac{r}{100}\right)$ rupees.

$\therefore$ The total *annual costs of lining*

$$= \frac{C}{Y} + \frac{C}{2} \times \frac{r}{100} \qquad ...(5.2)$$

Benefit cost ratio $= \dfrac{\text{Annual Benefits}}{\text{Annual Costs}}$

$$= \left[\frac{m \cdot R_1 + p \cdot R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}\right] \qquad ...(5.3)$$

If p is taken as 0.4, then

$$\text{Benefit cost ratio} = \left\{\frac{m \cdot R_1 + 0.4\, R_2}{\frac{C}{Y} + \frac{C}{2} \times \frac{r}{100}}\right] \qquad ...(5.4)$$

For project justification, benefit cost ratio must be greater than unity.

In addition to the benefits grouped above *i.e.* (water saving and reduction in maintenance cost) there may be benefits, like prevention of water–logging, reduced cost of drainage for adjoining lands, reduced risk of breaching, reduced incidence of malaria and other diseases in damp areas. Actual evaluation of these benefits is very difficult and may be approximated, based on experience, and may be taken into account for evaluating the annual benefit cost ratio.

Example 5.1. *An unlined canal giving a seepage loss of 3.3 cumecs per million sq. metres of wetted area is proposed to be lined with 10 cm thick cement concrete lining, which costs Rs. 180.00 per 10 sq. m. Given the following data, work out the economics of lining and benefit cost ratio.*

Annual revenue per cumec of water from all crops	*= Rs. 3.5 lakhs*
Discharge in the channel	*= 83.5 cumecs.*
Area of the channel	*= 40.8 sq. m.*
Wetted perimeter of the channel	*= 18.8 metres*
Wetted perimeter of the lining	*= 18.5 metres*
Annual maintenance cost of unlined channel per 10 sq. m.	*= Re. 1.00*

Assume additional suitable data, if required.

Solution.

Let us consider 1 km reach of canal.

Therefore, the wetted surface per km. $= 18.8 \times 1{,}000 = 18{,}800$ sq. m.

(*i*) **Annual Benefits**

(*a*) *Seepage.* Seepage loss in unlined canal @ 3.3 cumecs per million sq m

$$= \frac{3.3}{10^6} \times 18{,}800 \text{ cumecs/km.} = \left(\frac{62{,}040}{10^6}\right) \text{cumecs/km}$$

Assume seepage loss in lined channel at 0.01 cumec per million sq m of wetted perimeter.

$\therefore$ Seepage loss in lined canal

$$= \frac{0.01}{10^6} \times 18{,}800 = \frac{188}{10^6} \text{ cumecs/km}$$

$$\text{Net saving} = \left(\frac{62{,}040}{10^6} - \frac{188}{10^6}\right) = \frac{61{,}852}{10^6} = 0.06185 \text{ cumec/km}$$

Annual revenue saved per km of channel

$$= \text{Rs.} \frac{61852}{10^6} \times 3.5 \text{ lakhs} = \text{Rs. } 0.21648 \text{ lakhs} = \textbf{Rs. 21,648}$$

(*b*) *Saving in Maintenance*

Annual maintenance cost of unlined channel for 10 sq. m. = Re. 1.00

Total wetted perimeter per 1 km length = 18,800 sq. m.

$\therefore$ Annual maintenance charge for unlined channel per km = Rs. 1,880.00

Assume that 40% of this is saved in lined channel

Annual saving in maintenance charges = Rs. $0.4 \times 1{,}880.00$ = Rs. 752.00

$\therefore$ Total annual benefits per km = Rs. 21,648.00 + Rs. 752.00 = **Rs. 22,400**

(*ii*) **Annual Costs.** Area of lining per km of channel

$$= 18.5 \times 1{,}000 = 18{,}500 \text{ sq. m.}$$

Cost of lining per km of channel @ Rs. 180.00 per 10 sq. m.

$$= \text{Rs.} \frac{18{,}500 \times 180.00}{10} = \text{Rs. } 3{,}33{,}000$$

Assume life of lining as 40 years

$$\text{Depreciation cost per year} = \text{Rs.} \frac{3{,}33{,}000}{40} = \text{Rs. } 8325.00$$

Assume 5% rate of interest.

$$\text{Average annual interest} = \frac{1}{2} C \frac{r}{100} = \frac{1}{2} \text{Rs. } 3{,}33{,}000 \times \frac{5}{100} = \text{Rs. } 8325$$

Total annual cost = Rs. 8,325 + 8,325 = **Rs. 16,650**

$$\text{Benefit cost ratio} = \frac{\text{Annual benefits}}{\text{Annual costs}} = \frac{22{,}400}{16{,}650} = \textbf{1.35.}$$

B.C. Ratio is more than unity, and hence, the lining is justified.

Example 5.2. *An existing unlined channel is having the following dimensions :*

Width of the bottom	$= 1.52$ *m.*
Side slopes	$= 1\frac{1}{2} : 1$
Depth of flow	$= 0.91$ *metre*
Bed-slope	$= 0.0006.$

It is proposed to line this channel for the same discharging capacity. Find out the dimensions of the lined channel and work out the economics of the concrete lining, if following data are given :

Length of irrigation season = *150 days*
Saving in seepage loss by lining the canal = *1.5 per cent per km*
Cost of water = *Rs. 150.00 per hectare metre*
Cost of concrete lining = *Rs. 16.00 per sq. m.*
Cost of reshaping and trimming canal = *Rs. 4.00 per sq. m.*
Life of lining = *40 years.*
Interest rate = *7 per cent.*
Annual maintenance and operational cost (per km per year)
for unlined canals in earth = *Rs. 1000.00*
and for concrete lined canals = *Rs. 200.00*
Other additional benefits = *Rs. 350.00*

Assume any other necessary data, if not given.

Solution. Let us first of all work out the lined channel section which is required to pass the same discharge as is being passed by the given unlined channel.

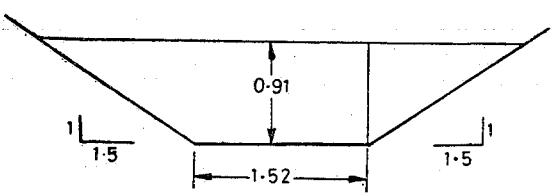


Fig. 5.1. Existing unlined canal section.

Discharge capacity of the existing unlined channel

If y is the water depth in channel, then

Sloping side $= y \cdot \sqrt{\left(\frac{3}{2}\right)^2 + (1)^2} = \frac{\sqrt{13}}{2} y = 1.8y = 1.8 \times 0.91 = 1.64$ m

Wetted perimeter $= 1.52 + 2 \times 1.8y = 1.52 + 3.6y = 1.52 + 3.6 \times 0.91 =$ **4.8 m**

Area $(A) = \left[\frac{1.52 + (3y + 1.52)}{2}\right] y = (1.52 + 1.5y)\, y = (1.52 + 1.5 \times 0.91)\, 0.91$

$= 0.91\,(1.52 + 1.36) = 0.91 \times 2.88 =$ **2.62 sq. m.**

$R = \frac{A}{P} = \frac{2.62}{4.8}$ m $= 0.546$ m.

By Manning's Formula

$$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

Assume n for unlined channel = 0.025

Velocity through unlined channel is given as :

$$V = \frac{1}{0.025} (0.546)^{2/3} \times (0.0006)^{1/2} = \textbf{0.654 m/sec.}$$

$$Q = A.V. = 2.62 \times 0.654 = \textbf{1.715 cumecs.}$$

In a lined channel, the water will flow more rapidly and the depth of flow will be less for the same discharge. Assume n for lined channel = 0.014.

Discharge through lined channel is given by :

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2} = \frac{1}{n} \cdot A \left(\frac{A}{P}\right)^{2/3} \cdot S^{1/2}$$

or $$1.715 = \frac{1}{0.014} \frac{(1.52y + 1.5y^2)^{5/3}}{(1.52 + 2 \times 1.8y)^{2/3}} \times (0.0006)^{1/2}$$

or $$1.715 = \frac{1}{0.014} \times \frac{2.45}{100} \left[\frac{(1.52y + 1.5y^2)^{5/3}}{(1.52 + 3.6y)^{2/3}} \right]$$

or $$\frac{1.715 \times 1.4}{2.45} = \frac{(1.52y + 1.5y^2)^{5/3}}{(1.52 + 3.6y)^{2/3}}$$

or $$0.98 = \frac{(1.52y + 1.5y^2)^{5/3}}{(1.52 + 3.6y)^{2/3}}.$$

Solving the above equation by hit and trial, we get

$$y = 0.67 \text{ m}.$$

If we use 0.15 m as free-board, then perimeter of lining is

$$P = 1.52 + 2 \times 1.8 \times 0.82 = 1.52 + 2.95 = \mathbf{4.47\ m}$$

Perimeter per km of lined channel $= 4.47 \times 1000 = \mathbf{4{,}470\ sq.\ m.}$

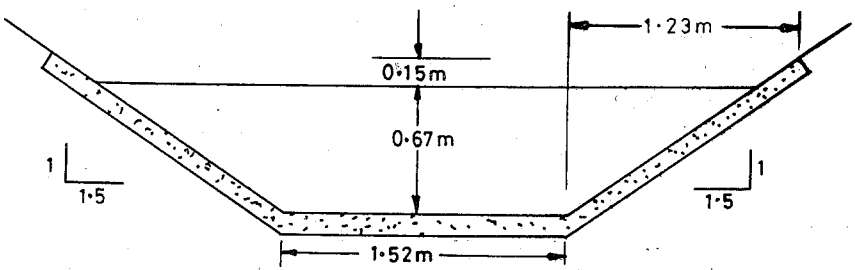


Fig. 5.2. Designed lined canal section.

(*i*) **Annual Benefits**

Seasonal flow. Discharge in the channel = 1.715 cumecs

1 cumec flowing for 150 days

$$= (1 \times 60 \times 60 \times 24 \times 150)\ \text{m}^3$$

1.715 cumecs flowing for 150 days

$$= (1.715 \times 3600 \times 24 \times 150)\ \text{m}^3$$

$$= \frac{1.715 \times 3600 \times 3600}{10^4} \text{ hectare–metre} = 2220 \text{ ha. m.}$$

$\therefore$ Seasonal flow = **2,220 hectare metres.**

Saving in seasonal seepage loss per km = 1.5% of flow

$$= \frac{1.5 \times 2220}{100} = \mathbf{33.3\ hectare\ metres.}$$

Money saved in seepage loss per km

$$= 33.3 \times \text{Rs. } 150 = \text{Rs. } 4995$$

Saving in maintenance charges

$$= \text{Rs. } 1000 - \text{Rs. } 200 = \text{Rs. } 800$$

Other benefits = Rs. 350

Total annual benefits = 4995 + 800 + 350 = **Rs. 6145**

(*ii*) **Annual Costs**

Perimeter of lining = 4,470 sq. m.

The cost of lining per km of channel @ Rs. 16 per sq. m.

$$= \text{Rs. } 16 \times 4{,}470 = \text{Rs. } 71{,}520$$

Life of lining = 40 years

$$\text{Annual depreciation charges} = \text{Rs.} \frac{71{,}520}{40} = \text{Rs. } 1788$$

Annual interest charges

$$= \frac{1}{2}\left(C \cdot \frac{r}{100}\right) = \frac{1}{2} \times 71{,}520 \times \frac{7}{100} = \text{Rs. } 2503$$

Total annual costs per km

$$= 1788 + 2{,}503 = \textbf{Rs. 4291} \qquad \ldots(ii)$$

$$\text{Benefit-cost ratio} = \frac{\text{Annual benefits}}{\text{Annual costs}} = \frac{\text{Rs. } 6145}{\text{Rs. } 4291} = \textbf{1.43.}$$

Benefit-cost ratio is sufficiently greater than unity, and hence, the lining is justified.

5.4. Justification for Lining Canals on New Projects

It has been stated earlier that the benefits obtained by lining canals in a new project are many more than what are obtained by lining the existing canals. The smaller canal sections, smaller earth work, smaller land acquisition, smaller and possibly fewer canal structures, reduced storage and diversion capacity, etc. are the additional benefits that are obtained in a new project.

These benefits may also be added in finding out the annual benefits for a new project.

DESIGN OF LINED IRRIGATION CHANNELS

Irrigation canals should be aligned and laid out, so that the velocity of flow is uniform under all conditions, and so that the water reaches the irrigated area at an elevation sufficient to ensure even and economical distribution. High velocities of flow can be permitted by taking the advantage of hard wearing surface, so as to ensure a hydraulically efficient channel.

Very high flow velocities, even if not damaging to lining, do entail extra expenditure at turnouts, and require higher walls to take care of pulsations or wave action. While aligning the channel, sharp curves should also be avoided, as they not only reduce the velocity of flow, but also require higher walls on the outside to retain the water as it rounds the curve.

5.5. Channel Cross-sections

Generally, two types of channel sections are adopted, *i.e.*

(*i*) Triangular channel section for smaller discharges, (*ii*) Trapezoidal channel section for larger discharges. In order to increase A/P ratio, the corners are rounded and attempts are made to use deeper sections by limiting depth, etc. The sections and their properties, most commonly used, are shown in Figs. 5.3 and 5.4.

(*i*) Triangular Section (Fig. 5.3)

Let central depth = radius of circle = y

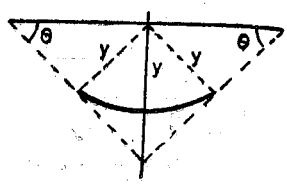


Fig. 5.3. Triangular section.

$$\text{Area} = \pi \cdot y^2 \cdot \frac{\theta}{\pi} + 2 \times \tfrac{1}{2} y \cdot y \cot \theta$$

$$= \frac{\pi \cdot y^2 \cdot \theta}{\pi} + y^2 \cot \theta$$

$$= y^2 [\theta + \cot \theta]$$

$$\text{Perimeter} = 2\pi \cdot y \times \frac{\theta}{\pi} + 2 \cdot y \cot \theta$$

$$= 2y \cdot \theta + 2y \cdot \cot \theta = 2y (\theta + \cot \theta)$$

Hence, Area for Fig. 5.3 $= y^2 (\theta + \cot \theta)$ ...(5.5)

Perimeter for Fig. 5.3 $= 2y (\theta + \cot \theta)$ ...(5.6)

Hydraulic mean depth for Fig. 5.3

$$= \frac{y^2 (\theta + \cot \theta)}{2y (\theta + \cot \theta)} = \frac{y}{2} \quad \text{...(5.7)}$$

(*ii*) Trapezoidal Section (Fig. 5.4)

$$\text{Area} = B \cdot y + 2 \cdot \left(\pi \cdot y^2 \frac{\theta}{2\pi} \right) + 2 \left(\tfrac{1}{2} y \cdot y \cot \theta \right)$$

or $$A = B \cdot y + y^2 \theta + y^2 \cot \theta$$

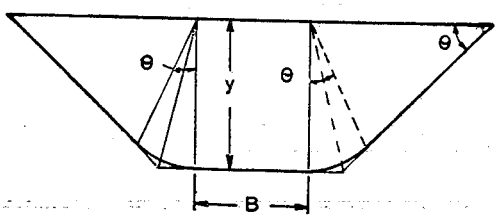


Fig. 5.4. Trapezoidal section.

Hence, area for Fig. 5.4

$$= \boxed{y (B + y\,\theta + y \cot \theta)} \quad \text{...(5.8)}$$

$$\text{Perimeter } (P) = B + 2 \left(2\pi \cdot y \cdot \frac{\theta}{2\pi} \right) + 2 \ (y \cot \theta)$$

Hence, perimeter for Fig. 5.4 = $\boxed{B + 2y\theta + 2y \cot \theta}$ ...(5.9)

The channel is, then designed, according to Manning's formula. The value of rugosity coefficient (n) depends upon the roughness of the channel boundary, and is different for different kinds of linings. Different values of n for different types of lining materials have already been tabulated in Table 5.2. The velocity or depth may be limited for designing a trapezoidal channel section.

5.6. Permissible Velocities in Lined Channels

Higher velocities can be safely used in lined channels. Though maximum permissible velocities for concrete linings have not been established, velocities up to 2.5 m/sec. (8 ft./sec) are permitted when the lining is not reinforced. The concrete lined Nangal Hydel Channel with a capacity of 355 cumecs is designed for a permissible average

velocity of 1.8 m/sec. (6 ft./sec.). If still higher velocities are desired, the lining can be reinforced accordingly. Asphaltic concrete, which has less resistance to abrasion, can withstand maximum velocity up to 1.5 m/sec. (5 ft./sec.). Values of limiting velocities in different kinds of linings, as per Indian Standards, are tabulated in Table 5.3.

Table 5.3. Max. Permissible Velocities in different types of Linings

Type of Lining	*Permissible Velocity*
Cement concrete lining (Unreinforced)	2.0 to 2.5 m/sec.
Burnt clay tile lining	1.8 m/sec.
Boulder lining	1.5 m/sec.

Example 5.3. *Design a lined channel to carry a discharge of 15 cumecs. The available and accepted country slope is 1 in 9000. Assume suitable values of side slopes and good brick work in lining.*

Solution. Let us first of all assume that the side slopes of the channel be $1\frac{1}{4} : 1$ (*i.e.* $1\frac{1}{4} H : 1\ V$) and the value of Manning's rugosity coefficient be 0.015 for good brick work. The channel section may be designed as triangular (as given in Fig. 5.3) because the discharge is small.

Considering Fig. 5.3, we have

$$\tan\theta = \frac{1}{1\frac{1}{4}} = \frac{1}{1.25}$$

or $\cot\theta = 1.25$

$\therefore$ $\theta = 0.675$ radians

Using $A = y^2(\theta + \cot\theta)$ ...(5.5)

and $P = 2y(\theta + \cot\theta)$ ...(5.6)

We get $A = y^2(0.675 + 1.25) = 1.925\, y^2$

$P = 2y(0.675 + 1.25) = 3.85\, y$

$$R = \frac{y}{2} = 0.5\, y \qquad \text{...(5.7)}$$

Now, using Manning's formula

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot S^{1/2}$$

Q is given to be 15 cumecs.

$$\therefore \quad 15 = \frac{1}{0.015} \times (1.925y^2) \cdot (0.5y)^{2/3} \frac{1}{\sqrt{9000}}$$

or $$15 = \frac{1.925}{0.015 \times 94.8} y^2 (0.63y^{2/3})$$

$$= 0.852\, y^{8/3}$$

$\therefore$ $y^{8/3} = 17.6$

or $y = (17.6)^{3/8} = \mathbf{2.93.\ metres.}$

Hence, use the section shown in Fig. 5.3 with 2.93 m depth and $1\frac{1}{4} : 1$ side slopes.

Example 5.4. *Design a concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 5,000. The side slopes of the channel may be taken as $1\frac{1}{2}$: 1. The value of n for lining is 0.014. Assume limiting velocity in the channel as 2m/sec.*

Solution. $$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

$$\therefore \quad 2 = \frac{1}{0.014} R^{2/3} \cdot \frac{1}{\sqrt{5000}}$$

or $$(2 \times 0.014 \times 70.8) = R^{2/3}$$

or $$R = (1.98)^{3/2} = \mathbf{2.79\ m.} \qquad ...(i)$$

The channel section is assumed to be trapezoidal as shown in Fig. 5.4. For $1\frac{1}{2}$: 1 slopes ; we have, $\cot \theta = 1.5$ or $\theta = 34.1° = 0.59$ radians.

Using $$A = y\,(B + y\theta + y \cot \theta) \qquad ...(5.8)$$

$$P = B + 2y\theta + 2y \cot \theta \qquad ...(5.9)$$

we get $$A = y\,(B + 0.59\,y + 1.5\,y) = y\,(B + 2.09\,y)$$

$$P = B + 1.18\,y + 3y \quad = B + 4.18\,y$$

But $$A = \frac{Q}{V} = \frac{350}{2} = 175 \text{ sq.m.}$$

$$\therefore \quad 175 = y\,(B + 2.09\,y)$$

or $$\frac{175}{y} = B + 2.09\,y$$

or $$\left(B = \frac{175}{y} - 2.09\,y\right) \qquad ...(ii)$$

From (*i*), we get

$$2.79 = R = \frac{A}{P} = \frac{175}{B + 4.18\,y}$$

or $$2.79 = \frac{175}{\left(\frac{175}{y} - 2.09y\right) + 4.18\,y}$$

or $$\left(\frac{175}{y} - 2.09\,y\right) + 4.18\,y = \frac{175}{2.79} = 62.7$$

or $$175 - 2.09y^2 + 4.18y^2 = 62.7y$$

or $$2.09\,y^2 - 62.7\,y + 175 = 0$$

or $$y^2 - 30\,y + 83.7 = 0$$

or $$y = \frac{30 \pm \sqrt{900 - 334.8}}{2} = \frac{30 \pm 23.8}{2} = \frac{30 - 23.8}{2}$$

(ignoring unfeasible + ve sign)

$$= \mathbf{3.2\ metres.}$$

But $$B = \frac{175}{y} - 2.09\,y = \frac{175}{3.2} - 2.09 \times 3.2 = 54.7 - 6.7 = \mathbf{48\ m.}$$

So use, Bed width = **48 m**
Depth = **3.2 m** **Ans.**

Example 5.5. *Design a concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 6400. The side slopes of the channel may be taken as $1\frac{1}{2}$: 1. The value of n for lining material may be taken as 0.013. Assume limiting depth of the channel as 4.0 m.*

Solution. The channel section is assumed to be trapezoidal as shown in Fig. 5.4. Now, we have

$$A = y\,(B + y\theta + y\cot\theta) \qquad ...(5.8)$$

$$P = B + 2y\cdot\theta - 2y\cdot\cot\theta \qquad ...(5.9)$$

For $1\frac{1}{2}$: 1 slope, $\cot\theta = 1.5$ and $\theta = 0.59$ radian.

$$\therefore \quad A = y\,(B + 0.59\,y + 1.5\,y) = y\,(B + 2.09\,y) \qquad ...(i)$$

$$P = B + 1.18\,y + 3y \qquad = B + 4.18\,y \qquad ...(ii)$$

$$y = 4.0 \text{ m (given)}$$

Therefore, from (i)

$$A = 4\,(B + 2.09 \times 4) = 4\,(B + 8.36) = \mathbf{4B + 33.44} \qquad ...(iii)$$

From (ii) $\quad P = B + 4.18 \times 4 = B + 16.72 \qquad ...(iv)$

$$R = \frac{A}{P} = \frac{4B + 33.44}{B + 16.72} \qquad ...(v)$$

$$Q = \frac{1}{n}\cdot A\cdot R^{2/3}\cdot S^{1/2}.$$

$$\therefore \quad 350 = \frac{1}{0.013} \times (4B + 33.44)\left(\frac{4B + 33.44}{B + 16.72}\right)^{2/3} \frac{1}{\sqrt{6400}}$$

or $$350 \times 0.013 \times 80 = \frac{(4B + 33.44)^{5/3}}{(B + 16.72)^{2/3}}$$

or $$364 = \frac{(4B + 33.44)^{5/3}}{(B + 16.72)^{2/3}} \qquad ...(A)$$

Solving Eq. (A) by Hit and Trial

(i) Use $\quad B = 30$ m.

$$\text{L.H.S.} = 364\ ;\ \text{R.H.S.} = \frac{(120 + 33.44)^{5/3}}{(30 + 16.72)^{2/3}} = \frac{(153.44)^{5/3}}{(46.72)^{2/3}} = \mathbf{339}$$

(ii) Use $\quad B = 32$ m.

$$\text{R.H.S.} = \frac{(161.44)^{5/3}}{(48.72)^{2/3}} = \mathbf{359}$$

(iii) Use $\quad B = 32.5$ m.

$$\text{R.H.S.} = \frac{(163.44)^{5/3}}{(49.22)^{2/3}} = 364;\ \therefore\ OK$$

Hence, use $B = \mathbf{32.5\ m}$, $y = \mathbf{4.0\ m}$ **Ans.**

TYPES OF LININGS, THEIR CONSTRUCTION, AND USES

Various types of canal linings, which are commonly adopted are enumerated below:

(*A*) **Hard Surface Linings**

(1) *Cast insitu Cement Concrete lining*
(2) *Shotcrete or Plaster lining*
(3) *Cement Concrete tile lining or Brick lining*
(4) *Asphaltic Concrete lining*
(5) *Boulder lining*

(*B*) **Earth Type Linings**

(1) *Compacted earth lining*
(2) *Soil cement lining*

All the above types of linings are discussed below :

5.7. Hard Surface or Rigid Linings

5.7.1. Cast Insitu Cement Concrete Lining. Cement concrete lining made from M15 cement concrete mix (1:2:4) is considered a *good quality type of lining.* Such linings usually give very satisfactory service, and are widely used owing to their durability, impermeability, hydraulic efficiency, and for providing weed free surface. Despite the fact that the initial cost of C.C. lining is high, yet its long life and minimum maintenance cost usually makes it an economical type of lining over the life span. For this reason, concrete linings have been extensively used in America and other developed nations. India being a developing nation, however, finds it difficult to afford their high initial costs, and as such, their use is restricted only to important projects. Concrete linings have been used in India in the canals of various important projects, such as Bhakra-Nangal Project, Tungabadhra Project, Amaravati Project, Sarda Canal in U.P., etc. About 570 km length of channels have been lined in concrete in Bhakra-Nangal Project.

5.7.1.1. Sub-grade preparations. Cement concrete linings are best suited for main canals which have to carry huge flows and at higher velocities. However, being rigid, its success depends to a great extent upon a stable foundation. Particular care should, therefore, be taken on thorough compaction of side slopes of the canals, on which the lining is to be laid. A firm foundation eliminates the possibility of cracking due to settlement of the subgrade. Natural earth in cutting, is usually, satisfactory ; but embankments in filling must be compacted by some suitable means.

Ordinarily, any rock or earth or predominantly sandy soil, is suitable as a subgrade material for cement concrete canal linings. However, when unusual subgrade conditions are encountered, such as expansive clays, susceptible to volume changes, then proper moisture and density control of the soil should be ensured during placing the lining. If the subgrade becomes too dry, it should be thoroughly wetted several hours before the lining is placed, so that the subgrade will be moist but not muddy at that time. Highly expensive soils like *black cotton soils*, need special treatment in the form of laying a layer of CNS (Cohesive non-swelling soil) material like *murum*, etc. in depth of about 300-1000 mm over the given soil surface before laying the lining.

Occasionally, it may be necessary to construct a concrete lined canal in areas where the ground water is likely to rise above the invert level of lining. In such cases, it may be necessary to lay drains underneath or alongside the canal, so as to relieve the hydrostatic pressure, which might cause uplift pressure on the lining during periods of low flow or no flow.

5.7.1.2. Thickness. A thickness of about 5 to 15 cm of cement concrete is generally used for larger canals, depending upon the canal capacity*, the nature of canal*, and the special requirements of imperviousness and structural strength to resist cracking, on slight movement of subgrade. *Minimum thickness used in India is 7.5 cm.* Stable side slopes of the order of $1\frac{1}{2}$: 1 or $1\frac{1}{4}$: 1 should be adopted. Steeper slopes entail extra earth pressure and consequently require more thickness, and hence are uneconomical.

5.7.1.3. Laying of Concrete The levelled and dressed sub-grade soil surface shall be moistened thoroughly before laying cement concrete, so that the moisture is not withdrawn by the sub grade from the cement concrete. In case an **inverted filter** is laid over the sub grade to take care of the differential hydrostatic pressure and draw-down in canals, it shall be properly designed keeping the coarsest filter material (gravel) layer in immediate contact with C.C. lining. *To make such filter blanket effective and to prevent ingress of concrete into it, a tar paper may be placed over the filter blanket before placing cement concrete.*

Hand placing of cement concrete is usually adopted in India, particularly for smaller canal sections. The cement concrete shall be dumped and spread on the bed and side slopes of the canal, usually in panels of not more than 3 m side, with suitable joints in between. The *construction joints* so formed will also serve the purpose of *contraction joints*, which are basically required for shrinkage of concrete and to take care of temperature stresses caused by the changes of temperature.

In order to prepare uniform panels, *screed guides* shall be laid on the sub grade, and the cement concrete shall be screeded up to the grade to proper thickness. Before laying the cement concrete for lining, precast cement concrete sleepers on side slopes and cast in situ C.C. sleepers on bed shall be placed under the joints, to serve as templates for accurate dressing of subgrade and to reduce the seepage through the joints. The sleepers shall be 20 cm wide and 10 cm deep for canals with capacity more than 15 cumec ; and 15 cm wide and 7.5 cm deep for canals with capacity less than 15 cumec. The sleepers shall be placed centrally below the joints. The C.C. used for sleepers shall

* *Hydel canals* oten require greater thickness of lining than the *irrigation canals*, because of the drawdown effects and non-possibility of closure of hydel canals for repairs. Similarly, deeper canals will have greater thickness than shallow depth canals. Minimum thickness of canal lining, based on canal capacity, is shown in table 5.4 below :

Table 5.4. Suggested Thickness of C.C. Lining for Irrigation Canals

S. No.	*Capacity of canal in cumec*	*Depth of water in m*	*Thickness of C.C. lining in cm*
1	0—5	0—1	5—6
2	5—50	1—2.5	6—7.5
3	50—200	2.5—4.5	7.5—10
4	200—300	4.5—6.5	9—10
5	300—700	6.5—9.0	12—15

be of the same grade as for lining. Mechanical Vibration of cement concrete is always preferred, for which, screed vibrators should be used.

Mechanical placing of cement concrete for laying C.C. lining is usually adopted on large sized projects in developed countries, by using *slip-form machines* supported on rails placed along both berms of the canal. Concrete for slip form should be *air-entrained* as to provide a more workable and slippable mix. The percentage of air recommended in shown in table 5.5.

Table 5.5. Percentage of air for Air-entrained cement concrete for C.C. lining to be placed by slip form machines

S. No.	*Maximum Aggregate size in mm*	*Air percent by volume*
1	10	8.0
2	12.5	7.0
3	20	6.0
4	25	5.0
5	40	4.5

For lining small to moderate sized canals, *sub grade guided slip forms* are used. The slip form is supported directly on the sub grade and operated longitudinally along it. Concrete is screeded on the bed along the canal length and on sides from bottom to top.

For larger canals of considerable length, *rail guided slip-forms* are used. Slip-forms supported on rails are placed along both berms of the canal and are operated longitudinally and on the side slopes from bottom to top.

For hand placing with the light machines, where concrete in screeded from bottom to the top of the side slope, the consistency of concrete should be such that it will barely stay on the slope. A slump of 60 to 70 mm is generally allowed. For heavier longitudinally operating slip-form machines, a slump of 50 mm at the laying point is permitted. There should be a close control on the consistency and workability of concrete, and the slumps should not vary more than 20 mm, otherwise there will be interference with the progress and quality of the work.

5.7.1.4. Finishing. The surface of concrete finished against forms shall be smooth and free from projections, honey-coming and other objectionable defects. Immediately on removal of forms, all unsightly ridges or lips shall be removed, and undesireable local bulging on exposed surfaces shall be remedied by tooling and rubbing. Repairs to concrete surfaces and additions where required shall be made by cutting regular openings into the concrete and placing fresh concrete to the required lines. The chipped openings shall be sharp and shall not be less than 70 mm in depth. The fresh concrete shall be reinforced and chipped and trowelled to the surface of the openings. The mortar shall be placed in layers not more than 20 mm in thickness after being compacted, and each layer shall be compacted thoroughly. All exposed concrete surfaces shall be cleaned of impurities, lumps of mortar or grout, and unsightly strains.

5.7.1.5. Curing. Subsequent to laying of concrete lining and after a period of 24-36 hours, the lining shall be cured for at least 28 days.

On bed, this may be done by constructing 150 mm deep earthen bunds across the bed, so that a small depth of water will regularly stand on the bed.

The curing of side slopes may be done by constructing masonry drains with weep holes or perforated pipes on the coping at the top of lining, or by sprinklers.

5.7.1.6. Surface drainage. Concurrently with the curing operation, surface drainage arrangement of the bank such as construction of dowels, bank surface slope away from the lining, and construction of longitudinal drain on the outer wedge shall be completed. This is necessary to prevent surface and subgrade erosion and consequent damage to the lining.

5.7.1.7. Joints in cement concrete lining. Cracks in C.C. lining usually occur due to :

(*i*) *Warping stresses* caused due to the difference in temperature between the atmosphere and the concrete lining, or due to moisture potential between the two faces of the C.C. slab ; and

(*ii*) *Tensile stresses* caused by the differential temperature variation between the upper and lower faces of the C.C. slab.

Cracking caused due to above causes is controlled either by providing reinforcement (as discussed in artile 5.7.1.8), or by providing joints in concrete lining, as discussed below :

The various types of joints which may be provided in C.C. lining are :

(*i*) Expansion joints ;

(*ii*) Construction joints ; and

(*iii*) Contraction joints.

Expansion joints are usually not provided in C.C. lining, except where structures intersect the canal. At the intersecting structure, an expansion joint of 25 mm width, filled with sealing compound, is provided.

Construction joints which are left during casting of cement concrete lining as pointed out earlier, do serve the purpose of **contraction joints**, which are specifically required to take care of shrinkage and temperature stresses. Each *construction joint* will oppose contraction stresses, and hence will be a *contraction joint*. Therefore, practically, there is no difference between the two types of joints. *Theoretically, however, the joints left due to difficulty in laying the cement concrete at a stretch, are called construction joints ; whereas, the joints left intentionally for making provision for shrinkage and temperature stresses, are called contraction joints.*

The most commonly adopted type of construction joint (serving as contraction joint also) provided while placing cement concrete lining in panels on top of C.C. sleepers, is shown in Fig. 5.5 (read in conjunction with Fig. 5.6 and table 5.6). Other types of construction joints, which are in use, are shown in Fig. 5.7.

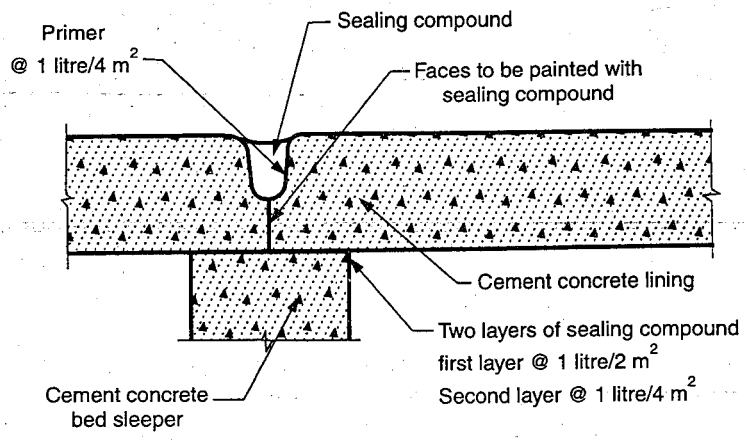


Fig. 5.5. Commonly adopted construction joints in cement concrete lining (IS : 3873-1978).

Table 5.6. Details of Groove to be provided in Expansion joint of Fig. 5.5 ; and lone contraction joints provided when lining is laid at a single stretch without leaving construction joints. (IS 3873-1978)

Values of distances b and c for various values of thickness (t) of lining (Fig. 5.6)			*App. groove spacing (c/c) m*
t (mm)	*b (mm)*	*c (mm)*	
50	8	17	3
65	8	20	3
75 and 80	11	27	4 to 5
90	11	30	4 to 5
100 and more	11	33	4 to 5

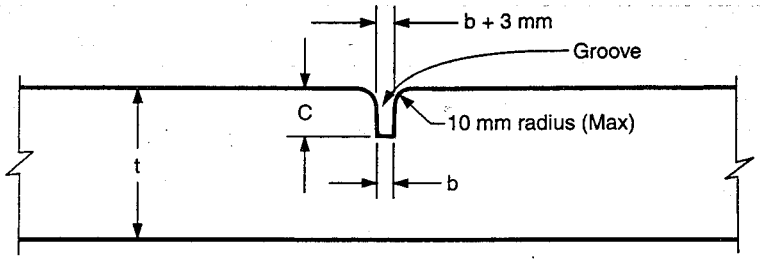


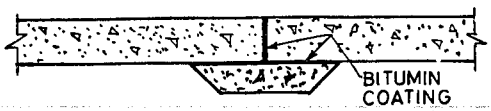
Fig. 5.6. Details of groove to form part of expansion joint of Fig. 5.5, or as an independent contraction joint.

Note 1. Allowable tolerance on *b* and *c* shall be ± 1.5 mm.

Note 2. The groove shall be cleaned and made free from foreign substances, and then filled with hot applied sealing compound, after curing period of cement concrete is over. During the curing period, course sand is filled in the groove which can be easily blown out while filling sealing compound.



(*a*) Lap joint.



(*b*) Lap joint with separate fillet.



(*c*) Tongue and groove joint.

Fig. 5.7. Various types of construction Joints in C.C. Linings.

5.7.1.8. *Reinforcement.* It has now been widely accepted that normal steel reinforcement (0.25 to 0.3% of concrete) adds practically nothing to the structural strength of uncracked lining. But it has been found that reinforcement reduces the width of shrinkage cracks, thereby reducing seepage and prevents possible faulting of the cracked slabs where unstable subgrades are encountered. In long slabs of more than 15 m or so, the intervals between transverse cracks was found to be more in unreinforced slabs as compared to those in reinforced slabs. If transverse joints can be provided at intervals sufficient to control intermediate cracking, the use of reinforcement is of no material advantage and is not justified. However, transverse expansion and contraction joints can be avoided by providing longitudinal steel reinforcement of the order of 0.5 per cent, and transverse steel reinforcement of the order of 0.25 per cent of the cross-sectional area of concrete. Such a lining will involve only the construction joints to be left after days work and longitudinal steel to be taken continuously through these joints.

Advantages of cast in situ cement concrete linings.

(*i*) Longer life than that of any other type.

(*ii*) Least permeable of all types. (*iii*) Most resistant to erosion.

(*iv*) Permits fast construction by mechanical means.

(*v*) Low recurring maintenance charges.

Disadvantages of cast-in-situ cement concrete linings.

(*i*) Higher initial cost. (*ii*) Greater possibility of temperature cracking.

(*iii*) Less flexible and easily affected by adverse subgrade conditions.

(*iv*) Skilled supervision and construction necessary.

Advantages of pre-cast cement concrete tile linings (discussed in article 5.7.3)

(*i*) Higher strength for equivalent thickness.

(*ii*) Avoids plaster finish. (*iii*) No lead of raw materials.

(*iv*) Lesser skilled labour, as campared to that required for cast in situ c.c. lining, is required for pre-cast c.c. tile lining.

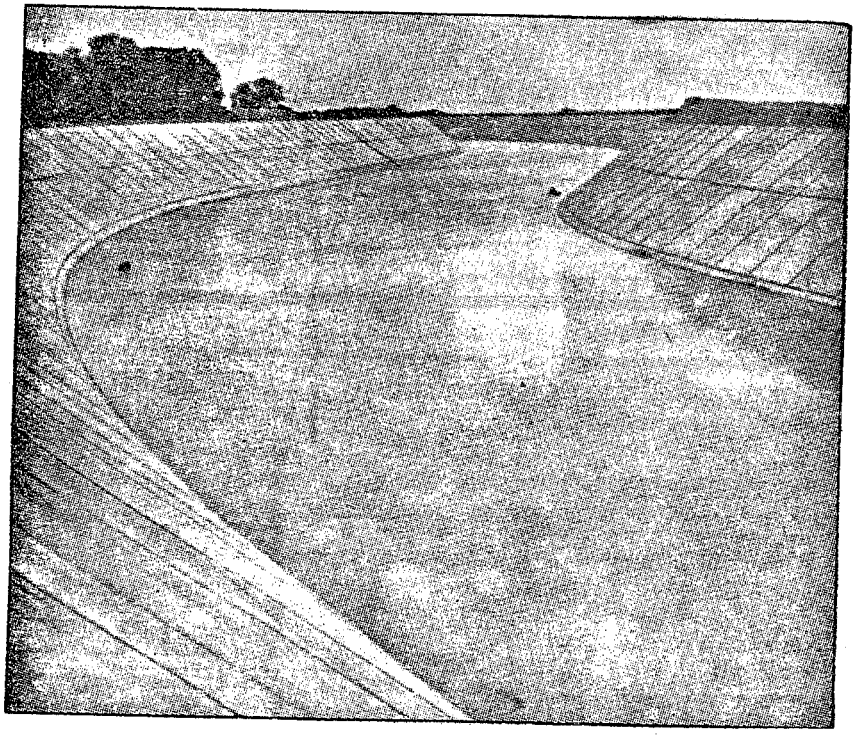


Fig. 5.8. Photographic view of one of the most unique artificial channels in the world, the Nangal power canal–constituting a part of the Bhakra-Nangal project–is lined with concrete throughout its length of 64 km.

(*v*) Easy to repair.

(*vi*) Various types of joints possible.

(*vii*) Lesser formwork than what is reqd for cas insitu c.c. lining, is required here.

Disadvantages of pre-cast cement concrete tile linings (discussed in article 5.7.3.)

(*i*) Slow progress.

(*ii*) Not suitable for curves.

(*iii*) Too light for hydel channels.

5.7.2. Shotcrete Lining. Shotcrete is a technical term used to designate cement mortar applied under pressure through a nozzle on the surface of the channel. It consists of a mixture of cement and sand (generally in the ratio of 1:4). Sand is having a maximum size of 0.5 cm. Larger proportions of cement are required in shotcrete as compared to what is required in cement concrete. Wire mesh reinforcement is generally, although not necessarily, used in shotcrete canal linings.

Shotcrete is of immense use for smaller jobs, because of the lighter equipment and small crew required to carry out the work. It is also useful for repair works and in rehabilitation of old canals. Shotcrete linings can be placed in an irregular canal section, thus eliminating the necessity of trimming the section, as is required for concrete canal

linings when placed with a slip-form. Shotcrete lining is generally laid in a thickness of about 3.5 cm.

Excavation, compaction, curing etc. for a shotcrete lining are the same as those required for a cement concrete lining.

5.7.3. Cement Concrete Tile Lining or Brick Lining. Such types of linings are very popular in India, because of certain inherent advantages in their use. These *advantages* are :

(*i*) Bricks or concrete tiles can be laid by ordinary masons, and specially skilled labour, as reqd for cast insitu c.c. lining, is not required.

(*ii*) Rigid quality control is not required.

(*iii*) No expansion joints are required.

(*iv*) Rounded sections can be easily laid without using any formwork.

(*v*) Larger number of labour is required, thus providing greater employment potential.

(*vi*) Isolated damaged portions can be repaired easily.

(*vii*) Bricks can be plastered to increase the carrying capacity of canal with the same section, and also to help increase the life span of lining.

The specifications may provide either a single or a double layer of c.c. tiles or bricks laid in mortar. Sometimes, a layer of tiles is laid over a layer of brick masonry. The top layer of tiles is generally laid in 1 : 3 cement mortar over 15 mm thick layer of plaster in 1 : 3 cement plaster. The size of tiles is generally restricted to $30 \times 150 \times 53$ mm. Typical sections are shown in Fig. 5.9 (*a*) and (*b*).

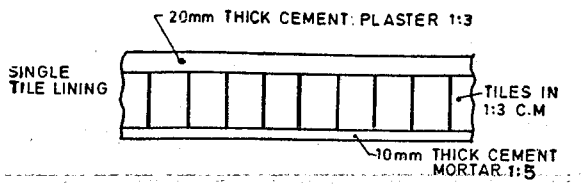


Fig. 5.9. (*a*) Single Tile Lining.

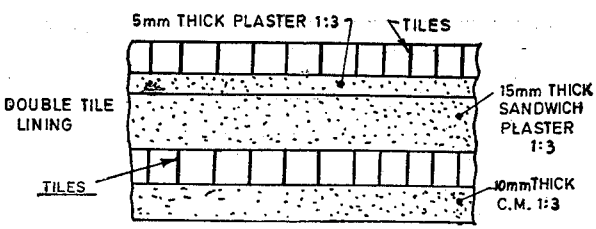


Fig. 5.9. (*b*) Double Tile Lining.

This type of lining gives a very satisfactory service. Even if there is a settlement of subgrade, the mortar joints between the bricks or tiles provide for numerous cracks so fine that seepage would be insignificant. In case of abnormal settlement, the local area can be repaired without any problem.

5.7.4. Asphaltic Concrete Lining. Asphalt has been used as a lining material at a very few places. It is still under the stage of evolution. Asphaltic concrete is a carefully controlled mixture of asphalt and graded stone aggregate, mixed and placed under elevated temperature. It provides a fairly cheap lining, especially where the asphalt is

available within the country. It is flexible and readily confirms to the subgrade. The disadvantages or the limitations of this type of lining are :

(*i*) It does not decrease the rugosity coefficient of the channel.

(*ii*) It permits certain type of weed growth.

5.7.5. Boulder Lining. Boulder lining, also called **dry stone lining** or **stone pitching**, consists of lining the side slopes* of on earthen canal by proper placement & packing of stones, either after laying a filter layer over the soil surface, or without any such filter, depending upon the site requirement. Such a lining does not prevent seapage of canal water, though helps in retaining the shape of canal section, thereby reducing maintenance cost. The stones to be used for lining are rounded or sub-angular river cobbles, or blasted rock pieces with sufficient base area, so as to remain stable in their position.

Stones are generally placed on the levelled subgrade, and hand packed after dividing side slope length into compartments by the construction of *Dhamalis (ribs)* of stone masonry constructed at suitable intervals. The dhamalis do rest on a c.c. or R.R. masoury toe wall constructed on the drain bed along the Junction of drain bed and side slope. Usually, dhamalis in R.R. masonry @15 m centre to centre, and 0.6 m in width, having depth equal to the depth of pitching (22.5 cm or so) are usually provided.

The biggest advantage of such a lining however, is that it is a **pervious lining** allowing free flow of water from the submerged or saturated subgrade into the canal. Such a lining, therefore, does not need any drainage arrangements, in the form of pressure relief valves, etc. as may be required in concrete or brick linings. Such a lining may therefore be preferred when watertable is very high in the area, higher than DBL or even FSL of the canal.

The thickness of the lining and the size of the stones, which may be adopted depending upon the canal capacity, are given in table 5.7.

Table 5.7. Thickness of stone lining to be adopted for different canal capacities

S. No.	*Canal/Drain capacity in cumecs*	*Thickness of stone lining recommended in cm.*	*Size of stones to be used*	
			Av. dia along longest axis in cm	*Min. dia. at any section in cm.*
1.	less than 50	15	15	7.5
2.	above 50 and less than 100	22.5	22.5	11.0
3.	100 and above	30	30	15.0

5.8. Earth Type Linings

5.8.1. Compacted Earth Lining. Soil graded to obtain the required characteristics and containing enough fines, so as to make it impervious, is thoroughly compacted at optimum moisture content**, and is used to provide a lining of 30 to 90 cm thickness. The use of this type of lining is restricted to the availability of suitable soils in the area,

* *Such* **dry stone lining** is generally adopted only on the side slopes of canals or drains, and the canal beds are rarely lined with such a lining.

** Please refer chapter 8 in "Soil Mechanics and Foundation Engineering" by the same author to know about compaction of earth under OMC.

through which the canal is being constructed. It would be uneconomical to transport the selected soils from outside.

5.8.2. Soil-Cement Lining. Portland cement up to the extent of 2 to 8% is added to the soil having a high percentage of fines. The mixture is first mixed dry. Water is then added so as to bring the soil to its optimum moisture content, and again mixed thoroughly. Material is then placed at site and compacted. Curing is then required for atleast seven days by covering with wet sand, etc.

5.9. Requirements of Good Lining

The canal linings should generally possess the following **essential requirements.**

(1) *economy ;* (2) *structural stability ;*
(3) *durability ; and* (4) *repairability,* as discussed below :

(1) **Economy.** The selection of a suitable type of lining for any canal project is mainly a question of economics and availability of *material, skilled and unskilled labour, construction machinery and equipment, and time available for completing the work.*

The type of lining selected should not only be economical in initial cost, but also in repair and maintenance cost.

Economic analysis of different types of linings, which can technically be used on a given project, is therefore of vital importance to us.

(2) **Structural stability.** The lining should be able to withstand the *differential sub-soil water pressure** from behind the lining due to sub-grade** getting saturated through seepage or rain or due to sudden drawdown of the canal. The lining should also be sufficiently heavy and strong to withstand the effect due to local cavity formation, if any, behind the lining.

(3) **Durability.** The canal lining should be able to withstand the natural wear and tear, such as the effect of *velocity of water, rain, sunshine, frost, thawing (applicable only in cold countries), thermal and moisture changes, and chemical action of salts,* etc. It should also be able to withstand the damaging effects caused by *cattle traffic, weed and rodent growth,* etc.

(4) **Repairability.** Since the lining will get damaged with its use over a period of time, it should be such that it can be repaired easily and economically. *Brick tile* or *concrete tile* or *stone boulder linings;* or *precast slab lining* can be easily repaired, as compared to *cast in situ concrete lining.*

In addition to the above *essential requirements,* the following may be the **additional requirements,** *depending on the need of a particular project:*

(*i*) *impermeability ;*

(*ii*) *hydraulic efficiency ; and*

(*iii*) *resistance to erosion,* as discussed below :

---

* Special drainage arrangements are provided for this, as discussed in article 5.11.

** Subgrade here means the soil surface of the ground against which the lining is placed on bed as well as sides of the canal.

(*i*) **Impermeability.** The permeability of lining may decide the quantum of seepage loss from a canal, which also is governed by the depth of water in the canal, and the type of subgrade soil. The permissible values of seepage losses from a canal for a particular area will depend upon the local conditions, such as the *values of land and water, population intensity,* etc. The lining chosen for a particular project should confirm to the allowable water losses.

(*ii*) **Hydraulic Efficiency.** The hydraulic efficiency of a canal, generally reduces with time, since the surface of lining gets eroded, increasing the friction factor (n), and thereby reducing its carrying capacity. The lining chosen for a given project must therefore, be such that its surface can be easily restored to original smoothness, if no reduction in its carrying capacity can be permitted. Cement plastered brick lining can be preferable in such a case.

(*iii*) **Resistance to erosion.** Sometimes, a canal may have to transport a considerable amount of sediment load, which may damage the lining by abrasion. Hence, in such a canal, the lining chosen should be able to withstand such abrasion. *Cement concrete and stone boulder linings may provide better abrasion resistance, as compared to brick tile lining.*

5.10. Factors Responsible for Selection of a Particular Type of Lining

Depending upon the project requirements as discussed above, the following factors may guide the selection of a particular lining for a given canal project :

(1) **Size and Importance of the canal.** For smaller canals, which may be used only intermittently, one may choose a lining, the construction of which may require little equipment and machinery. Larger canals on the other hand, may permit the use of cast in situ operations, with more elaborate T & P articles brought to site. Moreover, larger & important canals may require continuous operations, and, hence may need stronger linings, such as concrete lining.

(2) **Canal Slopes and Alignments.** These factors also need consideration since frequent changes in alignment and *steeper slopes* may encounter higher flow velocities, leading to selection of stronger linings. The limiting safe velocities, in generally used types of linings, are given in table 5.8, and may serve as a guide in this regard.

Table 5.8. Limiting Velocities in Different Types of Linings

S. No.	*Type of lining*	*Safe limiting velocity*
1.	Boulder lining	1.5 m/s
2.	Burnt clay tile lining	1.8 m/s
3.	Cement concrete lining	2.7 m/s*

(3) **Climate of the Area.** Higher quality linings should be used in areas which are susceptible to severe frosts and temperature changes such as in western countries.

* Other factors, like loss of head due to friction, would, however, limit the velocity in concrete lining to 2 m/s, or so.

(4) **Availability of Materials.** Every type of lining will require certain materials and ingredients. Some of these materials may be easily available at a certain place, and others may not be easily available. The type of lining should be such that the required materials are most easily available locally or in the vicinity of the area from where they can be carted to site with least cost.

(5) **Initial Expenditure.** Mathematically speaking, the most economical type of lining is the one which shows maximum annual benefit-cost ratio. This lining may have higher initial cost but longer life, than some other kind of lining having lesser value of annual benefit cost ratio. From long term planning point of view, the first type of lining should be chosen. But sometimes, the initial cost may be too high to be borne by the State, and hence, the lining with lesser initial cost may have to be adopted, even though its benefit-cost ratio may be less.

Keeping in mind the above factors, the suitable type of linings for different sizes of canals are given for *general guidance* in table 5.9.

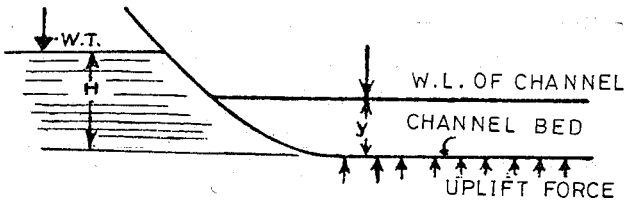
Table 5.9. General Guidelines for Choice of Linings for different sized Canals

S. No.	*Size of canal*	*Choice of Lining*
1.	Canals with bed width upto 3 m	(*i*) *Single burnt clay tile lining* ; or *brick lining* where seepage considerations are important.
		(*ii*) *P.C.C. slab lining* ; and
		(*iii*) *flexible membrane lining*, with adequate earth/tile cover.
2.	Canals with bed width between 3 to 8 m	(*i*) *single burnt clay tile lining* ;
		(*ii*) *P.C.C. slab lining* ; and
		(*iii*) *flexible membrane lining in the bed and rigid lining on the sides.* This may be adopted where channels have become stable and no danger of scour is expected.
3.	Canals with bed width greater than 8 m	(*i*) *insitu cement concrete lining in bed as well as on sides* ;
		(*ii*) *insitu cement concrete lining in bed and PCC slab lining on sides* ; and
		(*iii*) *burnt clay tile lining (single layer on bed and double layer on sides)* to be adopted at places where aggregates for manufacture of concrete are not available economically.

5.11. Under Drainage of Lined Canals (*i.e.* Drainage Behind Linings)

Many of the linings fail due to build up of water pressure behind the lining. Such a water pressure may essentially develop, when *the watertable rises above the canal bed* and there may be low flow or no flow in the canal. Such a situation will always cause an uplift on the lining, equal to the difference in the water heads, as shown in Fig. 5.10.

Such hydrostatic pressures may also be caused on the lining by the seeping rain water in the backfill, even *when the watertable remains below the canal bed, if the backfill is of low drainage* (*i.e.* of low permeability). In such a case, the backfill may



Uplift force = H metres of water
If $H > y$, a net uplift equal to $(H-y)$ metres of water will be exerted on channel bed. $(H-y)$ is called *Differential Head.*
Fig. 5.10.

get saturated over a period of time due to seepage of water through the joints and cracks; and in case of drawdown in the canal, the water in the backfill may not be drained out as quickly as the occuring drawdown, leading to building up of pressure behind the lining.

In all such cases, properly designed drainage arrangements must be provided, to help in reducing such hydrostatic pressures to safe limits. Such drainage arrangements may be provided in the form of *pressure release valves* for releasing the external water from the subgrade into the canal, through *drainage pockets* or through well connected *pipe drains* laid on the bed and sides of the canal, below the lining.

However, no such drainage arrangements may be necessary if the subgrade is made of clear gravel or sand of good permeability (more than 10^{-3} cm/sec or so) and the watertable is also not likely to go above the canal bed, because in such a case, there will be no appreciable time lag in the dissipation of drawdown pore pressures in the subgrade.

Drainage arrangement to be adopted will therefore depend upon : (*i*) *the position of the watertable ; (ii) upon the type of soil below the lining* ; and (*iii*) *the type of lining to be adopted.* Soil investigation is thus, of vital importance in designing such drainage arrangements.

5.11.1. Drainage Arrangements. Drainage arrangements, as stated above, may be accomplished by the following methods :

(1) **By providing drainage relief pockets**. Drainage relief pockets filled with *graded filter,* containing *gravel, coarse sand,* and *fine sand,* and provided with *pressure relief valves*, as shown in Fig. 5.11, may be used. The graded filter must, however, be properly designed on the considerations indicated below :

Design of graded filter. *The graded filter should be designed in such a way that there is no loss of bed soil. The gradation curve of the bed soil should be obtained from the sieve analysis. The D_{15} size of the adjoining layer A of the filter material should be atleast 4 times as large as D_{15} size of the bed soil, and less than 4 times its D_{85} size. Design of the other layers should be made in the similar way, till the requirement of the filter opening is met.*

Such *drainage relief pockets* may be provided at **isolated locations** in the bed as well as sides of the lined canal, below the lining at suitable spacings. They may be cubical

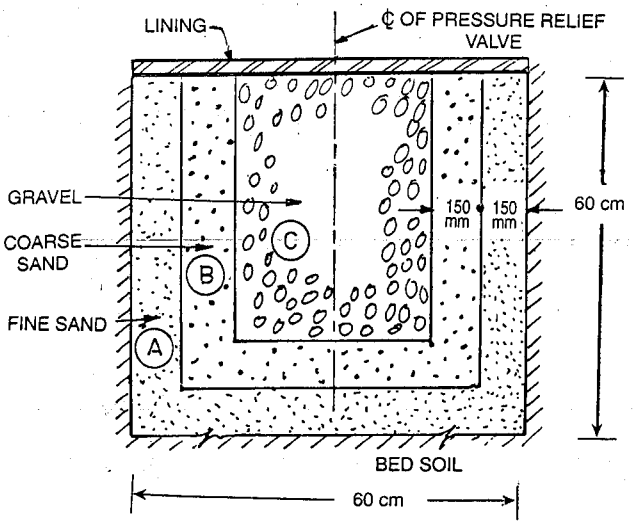


Fig. 5.11. Details of graded inverted filter to be used in drainage relief pockets.

in shape with 60 to 90 cm size, and may be provided @ 15 to 20 m intervals, along rows in such a way that atleast one row is provided on the bed for every 10 m bed width, and one row on sides for every 4 m side width, as shown in Fig. 5.12.

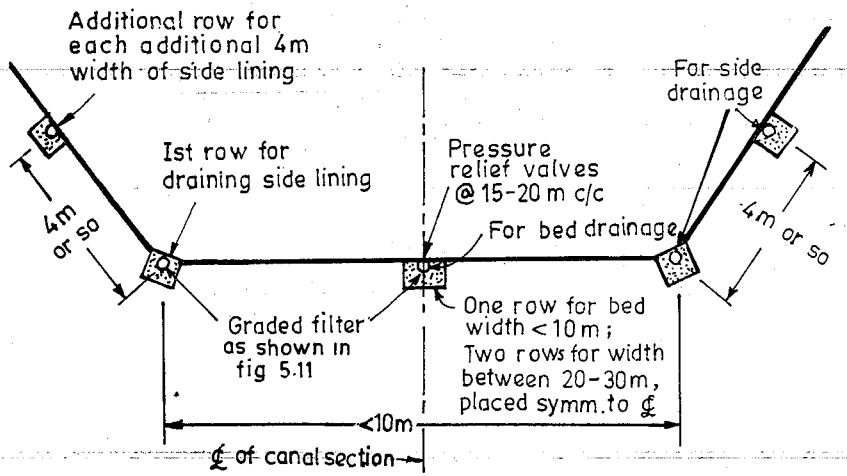


Fig. 5.12. Fig. showing locations of isolated drainage relief pockets.

Better drainage may sometimes be provided by providing **continuous drainage pockets** throughout the perimeter of the lined section of the canal, in 1 to 2 m width @ 15 to 20 m spacings, as shown in Fig. 5.13.

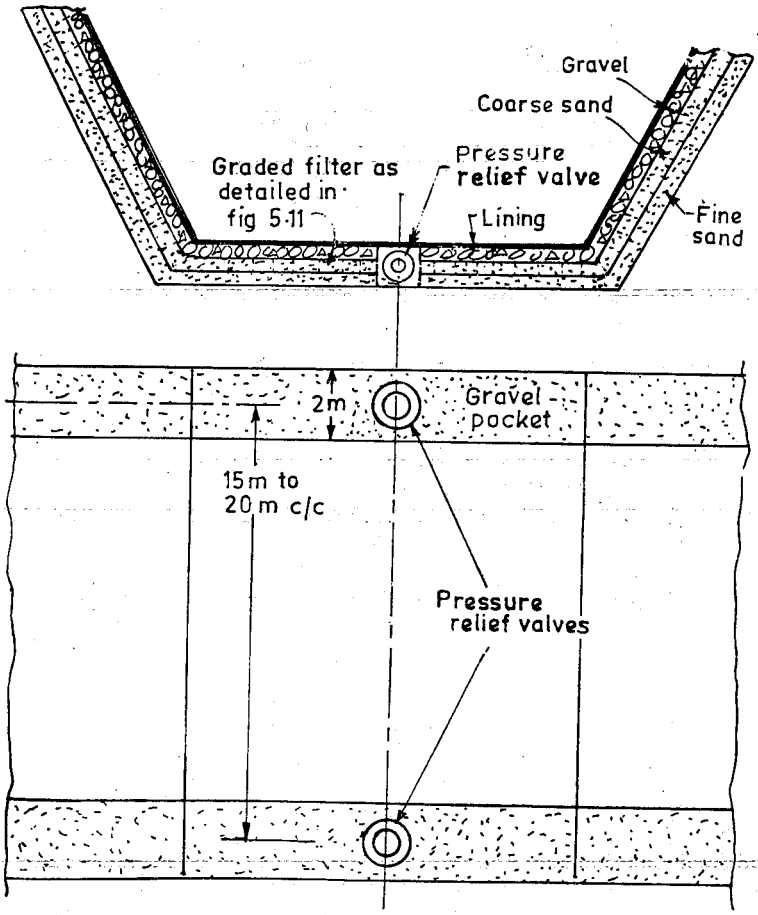


Fig. 5.13. Continuous drainage relief pockets running across the entire section in 1 to 2 m width at regular intervals along the lining length.

(2) By providing **open jointed pipe drains** in 50 cm × 50 cm sized trench, surrounded by graded gravel (5 to 20 mm in size) and discharging the drained water into the canal through pressure relief valves located at suitable intervals.

Such pipe drains will run logitudinally in a trench excavated below the lining on the canal bed along the length of canal, and transverse to the length of canal on the side slopes.

Water from the drains is collected in suitable masonry or precast concrete boxes, having pressure release valves placed on the top of the boxes, as shown in Fig. 5.14. This water is finally released into the canal through opening of the flap shutter of a pressure release valve, as and when the differential head exceeds 10 cm or so.

In case of **impervious subgrades,** these drainage arrangements may have to be further supplemented by laying sand or inverted filter, in say 75 cm thickness, underneath the entire lining.

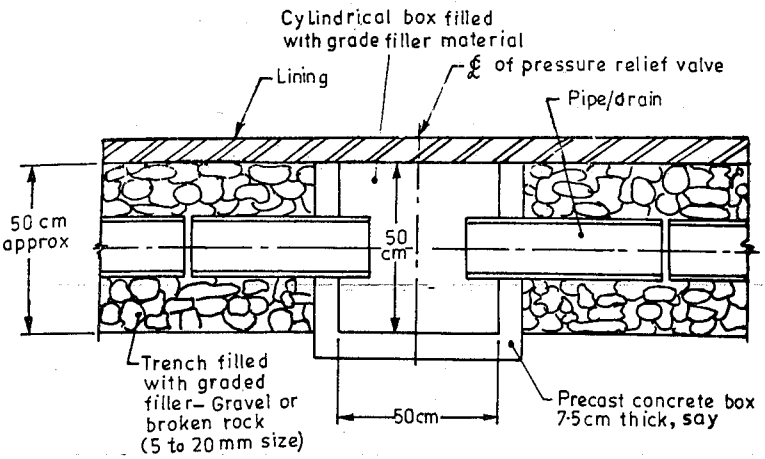


Fig. 5.14. Outlets for open jointed pipe drains.

5.11.1.1. Pressure relief valves. The flap valves, opening upwards into the canal, called pressure release valves, may help in releasing the hydrostatic pressure, as soon as the differential head exceeds the safe pressure for the given lining with a safety factor of at least 2. Generally, these valves open out, as soon as the differential pressure becomes 10 cm or so. These valves are available in the market in different sizes, say from 50 to 150 mm diameter size. 150 mm diameter valves may generally be used in the bed, and 50 mm diameter valves on side slopes. Typical details of both these types of valves are shown in Fig. 5.15 and 5.16, respectively.

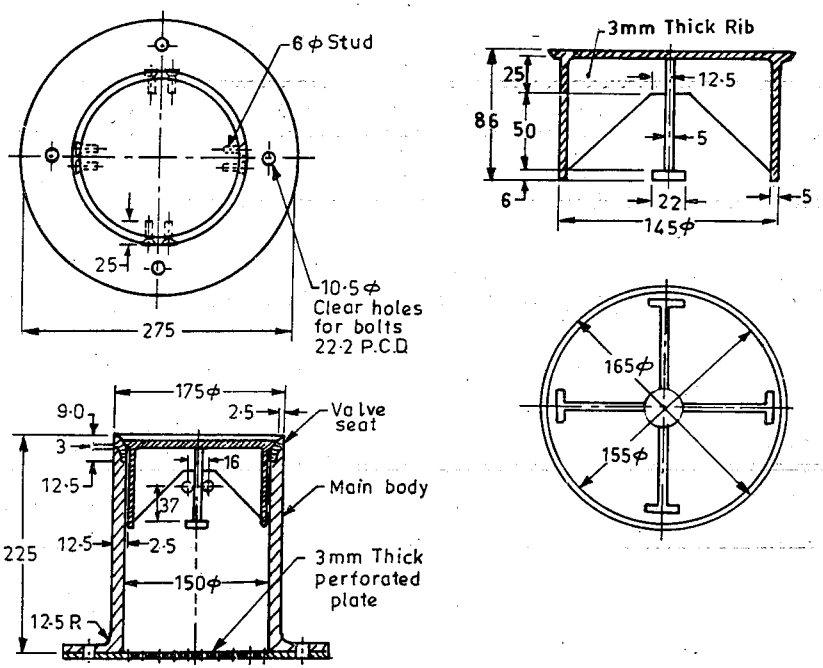


Fig. 5.15. Typical details of a 150 mm dia vertical pressure relief valve, useful on beds of lined canals.

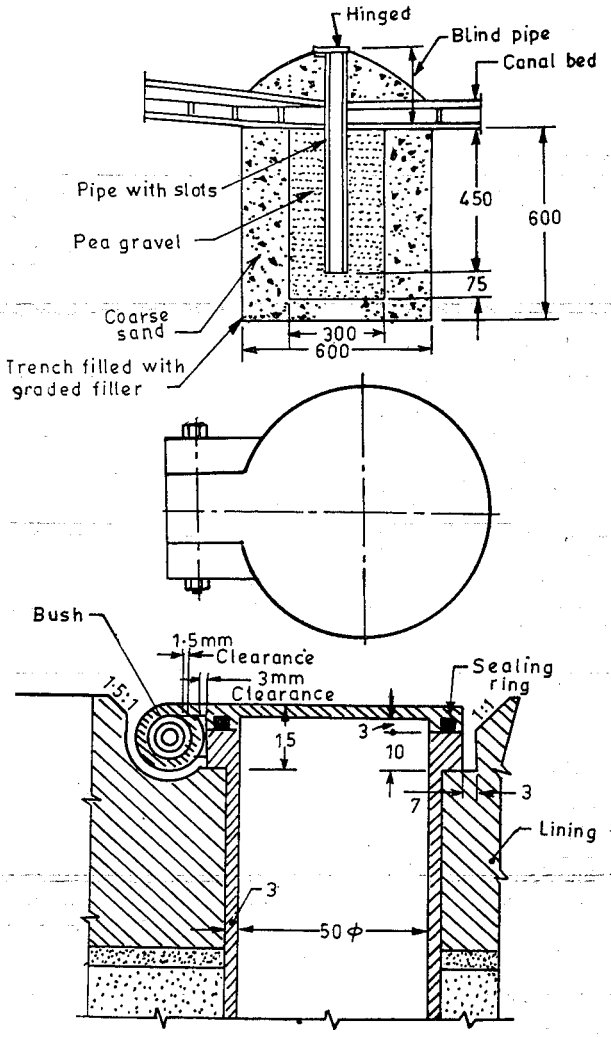


Fig. 5.16. Typical details of a 50 mm dia vertical pressure relief valve, useful on sides of lined canals.

These valves are generally made of such a material, which is resistant to abrasion and is not affected due to its submergence in water. Cast iron valves are generally available. Plastic valves are also making entry in the market with good potential.

5.12. Lining of Canals in Expansive Soils

Expansive soils are those soils, which exhibit *shrinkage* and *swelling* properties. Such soils are inorganic or organic clays of plasticity, with high compressibility, and liquid limits more than 50.

Commonly found expansive soils in India are called **black cotton soils**. Different varieties of black cotton soils, varying from *black* to *yellowish* to grey are found in our country. The chief property of such a soil is its *swelling pressure*. The larger is the swelling pressure, the more problemsome is the soil.

For canals excavated in such soils, lining is found to be very advantageous, because earthen canals in such soils pose serious problems of instability of slopes ; and it is thus, very difficult to maintain their regular sections.

But when lining materials are directly placed in contact with the expansive soils, they undergo deformations by *heaving,* disturbing the lining and throwing the canal out of commission. This deformation is dependent upon the swelling pressure developed by the given expansive soil, when it imbibes water in its intra-layers.

In order to counteract such swelling pressure, it is found advantageous to place a layer of some *cohesive non-swelling soil* (CNS)* like *muram* having some cohesion, over the given expansive soil surface. The larger the thickness of CNS introduced over the expansive soil, the lesser would be the resulting swelling and deformations.

Hence, in order to line canals in expansive soils, we generally place a layer of cohesive non-swelling soil over the soil subgrade, before placing the lining materials over the same.

The thickness of CNS to be used is more for more expansive soils and less for less expansive soils. Table 5.10 shows the CNS thickness to be adopted for soils of different swelling pressures. **

Table 5.10. Thickness of CNS Layer to be Used for Lining over Expansive Soil Sub-Grades

S. No.	*Swelling pressure of soil in KN/m²*	*Thickness of CNS material in cm**
1.	50—150	75 to 85
2.	200—300	90 to 100
3.	350 to 500	105 to 125

After providing a CNS layer, the lining can be done as usual. Precast cement concrete, burnt clay tiles, bricks, insitu cement concrete, or even boulder linings may be adopted. The under-drainage arrangements and joints in lining would also be provided, as usual.

5.13. Safety Ladders in Lined Canals

In large canals, safety ladders are generally provided on side slopes, at suitable intervals along the canal length. Such a ladder consists of a number of *ladder rungs,* constructed in canal lining at the given section, at different elevations, as shown in Fig. 5.17. The ladder rungs are made of smooth round mild steel bars, galvanised or coated with coaltar after installation, and are U-shaped as shown.

The ladders are provided on both banks, alternatively at about 300 metres-staggered distance, in straight reaches. Such ladders shall also always be constructed on both banks at about 30 m upstream of the point, where the canal enters some underground structure.

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* CNS material shall be non-swelling with a min. of 10 KN/m² (1020 kg/m²) of cohesion, and swelling pressure not more than 15 kN/m² at optimum moisture content. Most **murums** with some cohesive property perform setisfactorily as CNS material.

** Optimum thickness can be determined only be actual experiments in the field or the lab.

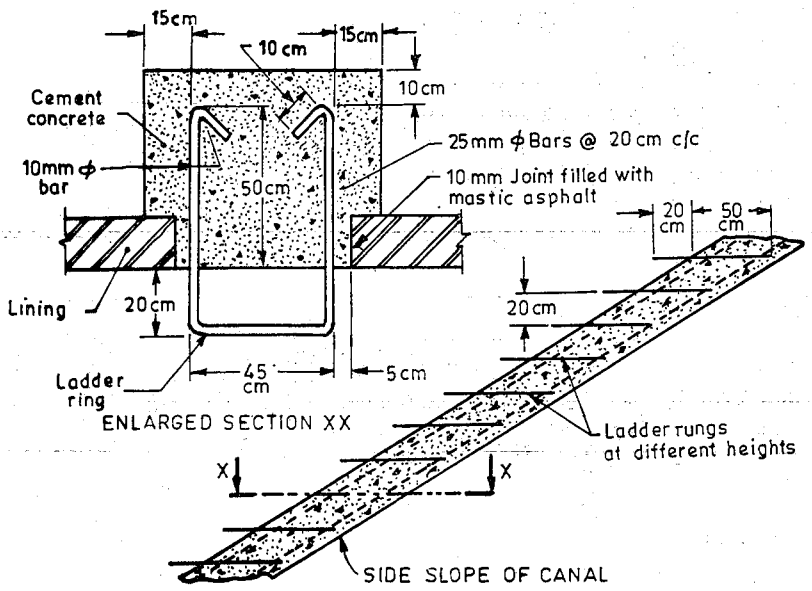


Fig. 5.17. Details of safety ladder.

These ladders can be used by persons or cattle, that may be swept away with the flowing water in the canal. Such a person, who may otherwise get drowned, may catch hold of one such ladder rung at the level of the flowing water, and can climb the higher rungs to get out of the canal, easily.

PROBLEMS

1. What is meant by 'canal lining', and what are its advantages ?

Enumerate the different types of canal linings, and discuss the design and construction features of concrete linings.

2. (*a*) Enumerate the different types of lining materials, and discuss the factors which are responsible for selecting a particular material in a particular project.

(*b*) Design a triangular concrete lined channel to carry a discharge of 20 cumecs at a slope of 10 cm/km. The side slopes of the channel are $1\frac{1}{4}$: 1. The value of n may be taken as 0.015.

[**Hint** : Follow example 5.3].

3. (*a*) Explain the necessity of lining of canals. Enumerate the various types of linings practised in Tamil Nadu State.

(*b*) Design a trapezoidal concrete lined channel to carry a discharge of 350 cumecs at a slope of 1 in 6400. The side slopes of the channel may be taken as $1\frac{1}{2}$: 1. The value of n for the lining material may be taken as 0.013. Assume the limiting B/D ratio to be 5. [**Ans.** Use $B = 20$ m, $D = 4$ m]

4. What are the main types of channel linings ?

Design a concrete lined trapezoidal channel to carry a discharge of 200 cumecs at a slope of 1 in 5000. The side slopes of the channel are 1 : 1 and Manning's coefficient of rugosity may be taken as 0.014. Assume the limiting velocity in the channel as 2 m per second.

[**Hint** : Follow example 5.4]

5. (*a*) How will you justify economically the necessity of lining an existing canal ? What added benefits you will expect if the canal to be lined is new and yet to be constructed ?

[**Hint** : Please see articles 5.3 and 5.4]

(*b*) What is meant by 'under-drainage of lined canals', and how is it provided ?

6. Write short notes on the following :

(*i*) Lining of irrigation canals. (*ii*) Under-drainage of lined canals.

(*iii*) Use of cement concrete for lining canals. (*iv*) Economics of canal lining.

(*v*) Various types of canal linings. (*vi*) Financial justification for lining new canals.

(*vii*) Advantages of lining irrigation canals. (*viii*) Safety ladders for large canals.

6

Reclaimation of Water-Logged and Saline Soils for Agricultural Purposes

6.1. Definition of Salinity and Water-logging

An agricultural land is said to be water-logged, when its productivity gets affected by the high watertable. The productivity of land infact, gets affected when the root zone of the plants gets flooded with water, and thus become ill-aerated. Ill aeration reduces crop yield, as explained below :

The life of a plant, infact, depends upon the nutrients like nitrates, and the form in which the nitrates are consumed by the plants is produced by the bacteria, under a process called *nitrification.* These bacteria need oxygen for their survival. The supply of oxygen gets cutoff when the land becomes ill aerated, resulting in the death of these bacteria, and fall in the production of plant's food (*i.e.* nitrates) and consequent reduction in the plant growth, which reduces the crop yield. Apart from ill-aeration of the plants, many other problems are created by water-logging, as discussed below :

(*i*) The normal cultivation operations, such as tilling, ploughing, etc. cannot be easily carried out in wet soils. In extreme cases, the free water may rise above the surface of the land, making the cultivation operations impossible. In ordinary language, such a land is called a swampy land.

(*ii*) Certain water loving plants like grasses, weeds, etc. grow profusely and luxuriantly in water-logged lands, thus affecting and interfering with the growth of the crops.

(*iii*) Water-logging also leads to salinity, as explained below :

If the watertable has risen up, or if the plant roots happen to come within the capillary fringe, water is continuously evaporated by capillarity. Thus, a continuous upward flow of water from the watertable to the land-surface, gets established. With this upward flow, the salts which are present in the water, also rise towards the surface, resulting in the deposition of salts in the root zone of the crops. The concentration of these alkali salts present in the root zone of the crops has a corroding effect on the roots, which reduces the osmotic activity of the plants and checks the plant growth, and the plant ultimately fades away. Such soils are called **saline soils.** From the above discussion, it becomes evident that the water-logging ultimately leads to salinity, the result of which is, the reduced crop yield. For this reason, salinity and water-logging are treated as a twin problem; under the head 'salinity and water-logging'. *Whenever there is water-logging, salinity is a must.*

6.2. Causes of Water-logging

Water-logging is the rise of watertable, which may occur due to the following factors :

(i) Over and Intensive Irrigation. When a policy of intensive irrigation is adopted, then, the maximum irrigable area of a small region is irrigated. This leads to, too much of irrigation, in that region, resulting in heavy percolation and subsequent rise of watertable. For this reason, to avoid water-logging, a policy of *extensive irrigation* (*i.e.* irrigation spread over wider regions) should supersede the policy of *intensive* irrigation.

(ii) Seepage of Water from the Adjoining High Lands. Water from the adjoining high lands may seep into the sub-soil of the affected land and may raise the watertable.

(iii) Seepage of Water through the Canals. Water may seep through the beds and sides of the adjoining canals, reservoirs, etc., situated at a higher level than the affected land; resulting in high watertable. This seepage is excessive, when soil at the site of canals, reservoirs, etc. is very pervious.

(iv) Impervious Obstruction. Water seeping below the soil moves horizontally (*i.e.* laterally) but may find an impervious obstruction, causing the rise of watertable on the upstream side of the obstruction. Similarly, an impervious stratum may occur below the top layers of pervious soils. In such cases, water seeping through the pervious soils will not be able to go deep, and hence, quickly resulting in high watertable.

(v) Inadequate Natural Drainage. Soils having less permeable sub-stratum (such as clay) below the top layers of pervious soils, will not be able to drain the water deep into the ground, and hence, resulting in high water level in the affected soil.

(vi) Inadequate Surface Drainage. Storm water falling over the land and the excess irrigation water should be removed and should not be allowed to percolate below. If proper drainage is not provided, the water will constantly percolate and will raise the level of the underground reservoir.

(vii) Excessive Rains. Excessive rainfall may create temporary water-logging, and in the absence of good drainage, it may lead to continued water-logging.

(viii) Submergence due to Floods. If a land continuously remains submerged by floods, water loving plants like grasses, weeds, etc. may grow, which obstruct the natural surface drainage of the soil, and thus, increasing the chances of water-logging.

(ix) Irregular or Flat Topography. In steep terrain, the water is drained out quickly. On flat or irregular terrain having depressions, etc., the drainage is very poor. All these factors lead to greater detention of water on the land, causing more percolation and raised watertable.

6.3. Water-logging Control

It is evident that water-logging can be controlled only if the quantity of water into the soil below is checked and reduced. To achieve this, the inflow of water into the underground reservoir should be reduced and the outflow from this reservoir should be increased, as to keep the highest position of water-table at least about 3m below the ground surface. The various measures adopted for controlling water- logging are enumerated below:

(1) Lining of Canals and Water Courses. Attempts should be made to reduce the seepage of water from the canals and water courses. This can be achieved by lining them. It is a very effective method to control water-logging.

(2) **Reducing the Intensity of Irrigation.** In areas where there is a possibility of water-logging, intensity of irrigation should be reduced. Only a small portion of irrigable land should receive canal water in one particular season. The remaining areas can receive water in the next season, by rotation.

(3) **By Introducing Crop-rotation.** Certain crops require more water and others require less water. If a field is always sown with a crop requiring more water, the chances of water-logging are more. In order to avoid this, a high water requiring crop should be followed by one requiring less water, and then by one requiring almost no water. Rice may be followed by wheat, and wheat may be followed by a dry crop such as cotton.

(4) **By Optimum Use of Water.** It is a known fact that only a certain fixed amount of irrigation water gives best productivity. Less than that and more than that, reduces the yield. But most of our cultivators are unaware of these technicalities, and they feel that by using more water they can increase crop yield. Therefore, they try to use more and more water. This can be checked by educating the cultivators by proper propaganda. Moreover, the revenue should not be charged on the basis of irrigated area but should be charged on the basis of the quantity of water utilised. A strict watch should also be kept at the outlet, in order to stop undue tapping.

(5) **By Providing Intercepting Drains.** Intercepting drains along the canals should be constructed, wherever necessary. These drains can intercept and prevent the seeping canal water from reaching the area likely to be water-logged.

(6) **By Provision of an Efficient Drainage System.** An efficient drainage system should be provided in order to drain away the storm water and the excess irrigation water. A good drainage system consists of surface drains as well as sub-surface drains (described in details a little later).

(7) **By Improving the Natural Drainage of the Area.** To reduce the percolation, the water should not be allowed to stand for a longer period. Some relief in this direction can be obtained by removing the obstructions from the path of natural flow. This can be achieved by removing bushes, jungles, forests, etc. and improving the slopes of the natural drainage lines.

(8) **By Adopting Consumptive Use of Surface and Subsurface Water.** The introduction of lift irrigation to utilize ground water helps in lowering the water-table in a canal irrigated area, where water-table tends to go up. *Hence, the ground water should also be used in conjunction with canal water for irrigation,* as the continuous use of ground water will not allow any appreciable rise in the level of water-table, due to continuous seepage of canal water.

This combined use of subsurface water (ground water) and the surface water (canal water) in a judicious manner, as to derive maximum benefits, called **conjunctive use**, should hence be adopted to control water-logging.

6.4. Reclamation of Saline and Alkaline Lands

Land reclamation is a process by which an unculturable land is made fit for cultivation. Saline and water logged lands give very less crop yields, and are, therefore, almost unfit for cultivation, unless they are reclaimed. Before summarising the remedies for reclaiming such lands, we shall first review the process, whereby, a land becomes, 'saline' or in extreme cases 'alkaline'.

Every agricultural soil contains certain mineral salts in it. Some of these salts are beneficial for plants as they provide the plant foods, while certain others prove injurious

to plant growth. These injurious salts are called *alkali salts* and their common examples are Na_2CO_3, Na_2SO_4, and NaCl. **Na_2CO_3 or black alkali* is the most harmful; and NaCl is the least harmful.** These salts are soluble in water. If the watertable rises up, or if the plant's roots happen to come within the capillary fringe, water from the watertable starts flowing upward. The soluble alkali salts also move up with water and get deposited in the soil within the plant roots as well as on the surface of the land. *This phenomenon of salts coming up in solution and forming a thin (5 to 7.5 cm) crust on the surface, after the evaporation of water, is called effeorescence.* Land affected by effeorescence is called *saline soil.* The salty water surrounding the roots of the plants reduces the osmotic activity of the plants, as explained below :

Since the plant roots act as semi-permeable membranes, so we have almost pure water on one side of the membrane (*i.e.* the water already extracted by the roots) and highly concentrated salt solution on the other side. Now, from the knowledge of physical chemistry, we can conclude that pure water from within the roots will start flowing out of the roots by **'osmosis'** towards the salt solution, until the pressure on pure water side becomes equal to the osmotic pressure of the salt solution. The plant will, hence, die due to lack of water, as shown in Fig. 6.1.

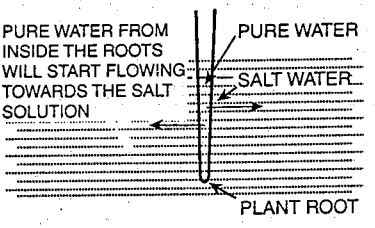


Fig. 6.1

Such a salt affected soil is unproductive and is known as "saline soil". If the salt effeorescence continues for a longer period, a base exchange reaction sets up, particularly if the soil is clayey, thus *sodiumising the clay*, making it impermeable and, therefore, ill-aerated and highly unproductive. Such soils are called *alkaline soils*. The reclamation of alkaline lands is more difficult.

Reclamation of salt affected lands. It is evident from the above discussion that effeorescence can be avoided if the watertable is maintained sufficiently (about 3m) below the roots, so that the capillary water is not able to reach the root zone of the plant. *Hence, all those measures which were suggested for preventing water-logging hold good for preventing salinity of lands also.* An efficient drainage system consisting of surface drains as well as sub-surface drains (explained in the next article) must be provided in order to control and lower the watertable in saline lands. After the high watertable has been lowered by suitable drainage, the soil is freed from the existing salts by a process, called *Leaching.*

6.4.1. Leaching. In this process, the land is flooded with adequate depth of water. The alkali salts present in the soil, get dissolved in this water, which percolate down to join the watertable or drained away by surface and sub-surface drains. The process is repeated till the salts in the top layer of the land are reduced to such as extent that some salt resistant crop can be grown. This process is known as leaching. High salt resistant crops like fodder, berseem, bajra etc., are now grown on this leached land for one or two seasons or till the salinity is reduced to such an extent that an ordinary crop like wheat, cotton, citrus garden crops, etc. can be grown. The land is then said to have been reclaimed.

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* It is known as black alkali because it dissolves some organic constituents of soil, which when in solution with it, appear black. The ground, therefore, gets spotted with patches of black stain.

When sodium carbonate (Na_2CO_3) is present in the saline soil, gypsum ($CaSO_4$) is generally added to the soil before leaching and thoroughly mixed with water. Na_2CO_3 reacts with $CaSO_4$ forming Na_2SO_4, which can be leached out as explained earlier.

6.4.1.1. Leaching requirement (LR) of a soil. In order to maintain status quo on the salinity of a given soil, and to avoid any further increase in its salinity, it is necessary to apply water to the soil in excess of the consumptive use (*i.e.* the requirement to meet evapotranspiration needs). This excess water will flow down beyond the root zone of the crop to the underground drainage system or to the underground reservoir, washing down the excess salts, which otherwise would have been deposited in the soil to further increase the salinity of the soil.

This excess water, which is required to meet the leaching needs, is generally expressed as the percentage of the total irrigation water applied to the soil (field) to meet the consumptive use as well as the leaching needs. This percentage quantity of water required for maintaining equilibrium in the salt content of the soil, has been computed to be expressed by the following equation :

$$LR\ (\text{LeachingRequirement}) = \frac{D_d}{D_i}$$

$$= \frac{\text{Depth of water drained out per unit area}}{\text{Depth of irrigation water applied per unit of area}} \qquad ...(6.1)$$

where D_i = Total irrigation water depth applied.

$$= \underset{\downarrow}{C_u} \quad + \quad \underset{\downarrow}{D_d}$$

$$\underset{\text{ptive use}}{\text{Consum-}} \quad + \quad \underset{\text{water depth}}{\text{Drained out}}$$

$$\therefore \quad L.R. = \frac{D_d}{D_i} = \frac{D_i - C_u}{D_i} \qquad ...(6.2)$$

For salt equilibrium, the ratio $\frac{D_d}{D_i}$ is found to be equal to $\frac{C_i}{C_d}$; where C_i is the salt content of irrigation water, and C_d is the salt content of drainage or leached water. Since the salt content is directly proportional to the Electrical conductivity* (EC), $\frac{C_i}{C_d}$ will be equal to $\frac{EC_{(i)}}{EC_{(d)}}$ where $EC_{(i)}$ is the *electrical conductivity of irrigation water*; and $EC_{(d)}$ is the *electrical conductivity of drained water* (leached water or leaching water). Hence, equation (6.1) can be written as :

$$L.R. = \frac{D_d}{D_i} = \frac{EC_{(i)}}{EC_{(d)}} \qquad ...(6.3)$$

The *E.C.* of drainage water, or leaching water, *i.e.* $EC_{(d)}$, may be assumed on the basis of permissible salt tolerance limit of the grown crop, but is generally assumed to

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* **Electrical conductivity** is a measure of salt content in a given water sample, and is dealt in more details in article 1.7(2).

be twice the E.C. value of the *saturation soil extract** *i.e.*, $E.C._{(e)}$. Hence, eqn. (6.3) can also be written as :

$$L.R. = \frac{D_d}{D_i} = \frac{EC_{(i)}}{EC_{(d)}} = \frac{EC_{(i)}}{2EC_{(e)}} \qquad ...(6.4)$$

Example 6.1. *Estimate the leaching requirement when electrical conductivity (EC) value of a saturated extract of soil is 10 m mho/cm at 25% reduction in the yield of a crop. The EC of irrigation water is 1.2 m.mho/cm. What will be the required depth of water to be applied to the field if the consumptive use requirement of the crop is 80 mm? EC value of the leaching water may be suitably assumed.*

Solution. The given values are :

$EC_{(e)}$ = E.C. value of saturated soil extract = 10 milli mho/cm

$EC_{(i)}$ = E.C. value of irrigation water = 1.2 milli mho/cm

C_u = Consumptive use = 80 mm

The *Leaching Requirement* (LR) is given by the Eqn. (6.3) as :

$$LR = \frac{D_d}{D_i} = \frac{EC_{(i)}}{EC_{(d)}}$$

where $EC_{(d)}$ is the E.C. value of leaching water, which may be assumed to be equal to $2 \cdot EC_{(e)} = 2 \times 10$ m.mho/cm = 20 milli mho/cm

Substituting the above values, we get

$$L.R. = \frac{EC_{(i)}}{EC_{(d)}} = \frac{1.2 \text{ milli mho/cm}}{20 \text{ milli mho/cm}} = \frac{1.2}{20} \times 100\% = 6\%$$

Hence, the Leaching Requirement is 6%. **Ans.** ...(1)

Now using eqn. (6.2), we have

$$LR = \frac{D_d}{D_i} = \frac{D_i - C_u}{D_i} = \frac{D_i - 80 \text{ mm}}{D_i} \times 100\% \qquad ...(2)$$

Equating (1) and (2), we have

$$6 = \left(\frac{D_i - 80 \text{ mm}}{D_i}\right) \times 100$$

or $6 D_i = 100 D_i - 8000$ mm or $94 D_i = 8000$ mm

or $D_i = \frac{8000}{94}$ mm = **85.1 mm**

Hence, the required water depth for irrigation = **85.1 mm Ans.**

LAND DRAINAGE

Surface irrigation is a blessing only if it is practised with great care. Only optimum amount of water should be supplied to the crop, in accordance with the requirement of that crop, and the properties of the soil must be given full consideration. Excess water, which the root zone of the soil fails to absorb, may percolate and help in raising the

* The water solution extracted from a soil at its saturation percentage.

watertable. Sometimes, this gravity water may encounter an impervious stratum and may not be drained up to the watertable. As explained earlier, this excess water is not only a waste but may be harmful to crop yield also. If such conditions are likely to occur, it becomes necessary that the excess water is removed and drained out from below the soil and discharged back either into a river, a canal, etc. or somewhere else. *Hence, while designing a canal irrigation network, it is sometimes desirable to provide a suitable drainage system, for removing the excess irrigation water.* This may be necessary in areas of high watertable and in river deltas, when irrigation facilities are extended to such areas. Drainage system is also required for draining out the storm water, and thus to prevent its percolation and to ensure easy disposal. Two types of drainage can be provided, *i.e.*,

(1) Surface drainage, (2) Sub-surface drainage, called Tile- drainage or Underground drainage. These are explained below :

6.5. Surface Drainage or Open Drainage

Surface drainage is the removal of excess rain water falling on the fields or the excess irrigation water applied to the fields, by constructing open ditches, field drains, and other related structures. The land is sloped towards these ditches or drains, as to make the excess water flow in to these drains.

When irrigation is extended to arid regions, drainage ditches become necessary to remove water required for leaching undesirable salts from the soil, and to dispose off the excess rainfall.

The open drains, which are constructed to remove the excess irrigation water collected in the depressions on the fields, as well as the storm (rain) water, are broad and shallow, and are called *shallow surface drains.* These drains carry the runoff to the *outlet drains,* which are large enough to carry the flood water of the catchment area from the shallow surface drains, and are of sufficient depths to provide outlets even for the underground tile drains, if provided. These outlet drains may be called *deep surface drains.*

Surface drains constructed for removing excess irrigation water applied to the farms and the storm water, cannot and should not be deep enough, as to interfere with the agricultural operations. They are, therefore, designed as shallow surface drains.

Land grading, which results in a continuous land slope towards the field drains, is an important part of a surface drainage system. Land grading or land leveling is also necessary for surface irrigation.

The **shallow surface drains** are trapezoidal in cross-section. Strictly speaking, they should be designed to carry the normal storm water from the fields, plus the excess irrigation water. Many a times, the excess irrigation water is neglected and these drains are designed only for the runoff resulting from the average storms. It is neither economical nor desirable to design these drains for exceptional storms. Kutter's or Manning's equations may be used to design these drains, keeping the velocity within the limits of the critical velocity, and thereby avoiding silting or scouring. Manning's equation is, however, generally used for the design of shallow as well as deep surface drains.

Deep surface drains or **outlet drains** carry the storm water discharge from the shallow surface drains, and the seepage water coming from the underground tile drains. They are, therefore, designed for the combined discharge of the shallow surface drains

as well as that of the tile drains. Generally, a cunnette of about 0.6 m depth is provided in the centre of the drain-bed, so as to carry the seepage water of the underground tile drains. A steeper slope is given to the cunnette and it is lined, so as to withstand higher flow velocities, and thus, to inhibit weed growth. The full section would be operative only during the rainy season, as otherwise, the flow will be confined only within the cunnette.

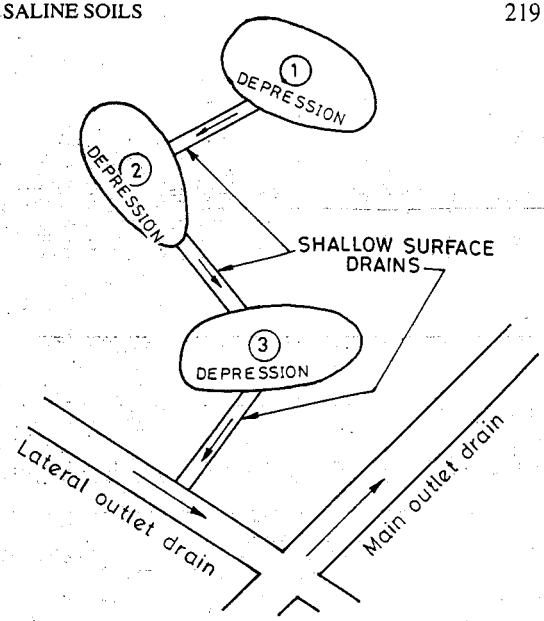


Fig. 6.2. Random field-drain (shallow surface drain) system for surface drainage.

6.5.1. Surface Inlet. The surface water from the pot holes, depressions, road ditches, farm steads, etc. may be removed either by connecting them with the shallow surface drains, sometimes called *random field drains* (shown in Fig. 6.2), or by constructing an intake structure called an *open inlet* or *surface inlet* (Fig. 6.3). A *surface inlet* is a structure constructed to carry the pit water into the sub-surface or tile drain. A cast iron pipe or a manhole constructed of brick or monolithic concrete, is sufficient and satisfactory. Manholes with sediment basins are sometimes used as surface inlets.

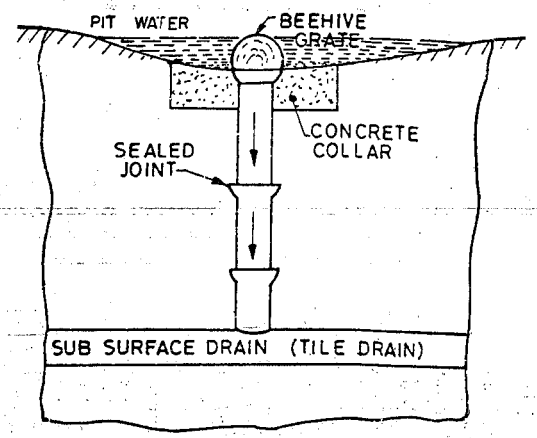


Fig. 6.3. surface inlet draining the surface water into a tile drain.

At the surface of the ground, a concrete collar extending around the intake is constructed on the riser to prevent growth of vegetation and to hold it in place. On the top of the riser, beehive grate or some other suitable grate is provided, so as to prevent trash from entering the tile. When the inlet is constructed in a cultivated field, the area immediately around the intake should be kept in grass.

When the surface inlet is connected to a main tile drain, it is a good practice to offset the surface inlet from the main. Such constructions may eliminate failure of the system, if the surface inlet structure should become damaged.

6.5.2. French Drain. When the quantity of water to be removed from the pits or depressions is small, a *blind inlet* may be installed over the tile drain. The blind inlet is also called *french drain.* These are constructed by back filling the trench of the tile drain with

graded materials, such as gravel and coarse sand, or with corn cobs, straw and similar substances, as shown in Fig. 6.4.

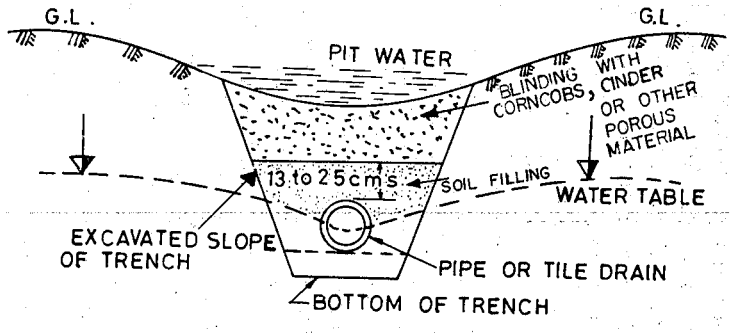


Fig. 6.4. Blind inlet or French drain.

Such inlets are not permanently effective. The voids in the backfill of the blind inlet become filled up with the passage of time, thereby reducing its effectiveness. Even though they are not permanently effective, they are economical to be installed and do not interfere with the farming operations.

6.5.3. Bedding. Bedding is a method of surface drainage which makes use of dead furrows, as shown in Fig. 6.5. The area between the two adjacent furrows is known as

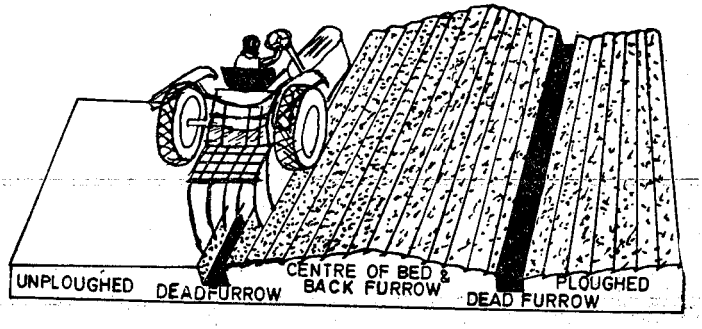


Fig. 6.5. Cross-section of bed showing method of construction.

a bed. The depth of the bed depends on the soil characteristics and tillage practices. In the bedded area, the direction of farming may be parallel or normal to dead furrows. Tillage practices, parallel to the beds, retard water movement to the dead furrows. Ploughing is always parallel to the dead furrows. Bedding is most practicable on flat slopes of less than 15%, where the soils are slowly permeable and the drainage is not economical.

6.6. Sub-surface Drainage or Tile Drainage

Plants need air as well as moisture in their root zones for their survival. Excess irrigation farm water is free to move into the underground tile drains, if provided*. This water, if not removed, retards the plant growth, because it fills the soil voids and restricts proper aeration. *Surface drains are, therefore, needed for removing the excess farm*

* Underground tile drains are not very common in India, where water-tables are generally falling and availability of water is becoming scarce.

water, for most of the cultivated crops on flat or undulating topography. Sub-surface drains, on the other hand, are required for soils with poor internal drainage and a high watertable. If no impervious layer occurs below the farm land and the watertable is low (lower than about 3m from the ground), internal soil drainage may be sufficient and no tile drains needed. For maximum productivity of most of the crops, both surface as well as sub-surface drains may sometimes,however, become, essential, particularly in areas of higher water-tables.

Advantages of Tile Drains. Tile drainage helps in increasing crop yields by draining the water or by lowering the watertable in the following manner :

(*i*) Removes the free gravity water that is not directly available to the plants.

(*ii*) Increases the volume of soil from which roots can obtain food.

(*iii*) Increases air circulation.

(*iv*) Increases bacterial activity in the soil, thus improving soil structure and making the plant food more readily available.

(*v*) Reduces soil erosion. A well drained soil has more capacity to hold rainfall, resulting in less runoff and hence, reduced erosion.

(*vi*) Reduces and removes toxic substances such as sodium and other soluble salts, which when present in large concentrations may retard plant growth.

(*vii*) Lesser time and labour is required for tilling and harvesting the soils, as these drains do not obstruct farming operations. With a crop such as corn, a delay in planting may decrease the yields. Planting in wet soils is also likely to decrease yields. All such troubles are removed in tile drained soils.

(*viii*) Tile drains permit deep roots development by lowering the watertable, especially during spring months, as shown in Fig. 6.6.

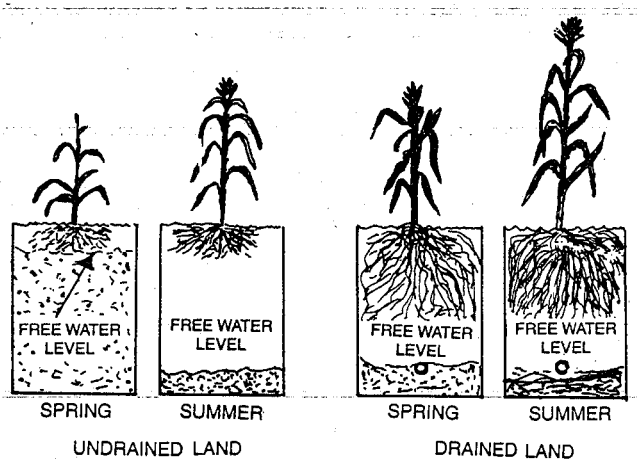


Fig. 6.6. Root development of crops grown on drained and undrained land.

Note : A plant having deep roots can extract water from greater depths and hence, can withstand droughts better than the one having shallow roots. Moreover, a deep rooted plant is larger and, therefore, capable of more transpiration and hence, giving increased yields.

Providing underground tile drains, however, is a costly proposition and may be required only in areas of high water-table, and where the ground soil has a poor internal drainage capacity.

6.6.1. Envelope Filters. Tile drains, are usually, pipe drains made up of porous earthenware and are circular in section. The diameters may vary from 10 to 30 cm or so. These drains are laid below the ground level, butting each other with open joints. The trenches in which they are laid, are back filled with sand and excavated material, as shown in Fig. 6.7. As far as possible, the tile drains should not be placed below less pervious strata. Because in that case, they may remain dry even though the land above the impervious strata may be water-logged, as the water will not be able to reach the drain.

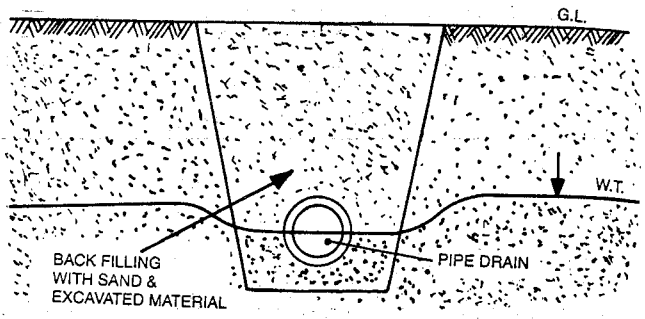


Fig. 6.7. Cross-section of a tile drain in pervious soils (without any filter).

When tile drains are placed in less pervious soils, they are generally surrounded by graded gravel filters, called *envelope filters* (Fig. 6.8). The envelope filter serves two functions : (*i*) it prevents the inflow of the soil into the drain, and (*ii*) it increases the effective tile diameter, and thus increases the inflow rate. The filter consists of different gradations, such as gravel, coarse sand, bajri, etc., The coarsest material is placed immediately over the tile, and the size is gradually reduced towards the surface. The minimum thickness of the filter is about 7.5 cm. The graded filter may sometimes be substituted by a single gradation, depending upon the availability and cost considerations.

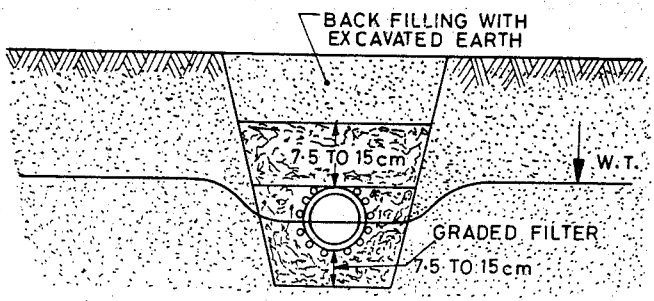


Fig. 6.8. Cross-section of a tile drain in less pervious soils (*with graded filter*).

6.6.2. Outlets for Tile Drains or Closed Drains. The water drained by the tile drains is discharged into some bigger drains, called deep surface drains. The water from a tile drain may be discharged into an outlet drain either by gravity or by pumping, depending upon which, we can have gravity outlet or pump outlet, as described below:

(*a*) **Gravity outlet.** If the bed level and the full supply level (FSL) of the outlet drain is lower than the invert level of the tile drain, then the water can be discharged

easily into the outlet drain by the mere action of gravity. Corrugated metal pipe with a flap shutter to prevent entry of rodents and back flow from the outlet drain into the tile drain, is generally provided at the outfall point, as shown in Fig. 6.9.

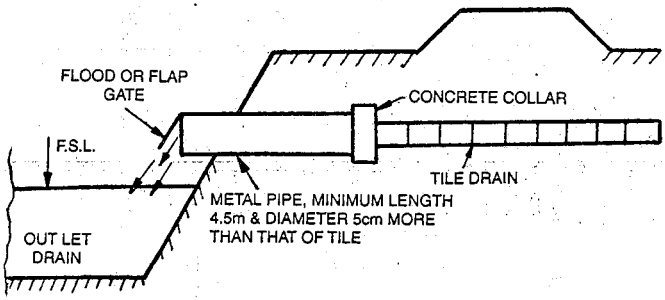


Fig. 6.9. Gravity outlet (outfall) for tile drain.

(*b*) **Pump outlet.** When the bed level of the outlet drain is higher than that of the discharging tile drain; a pump outlet has to be installed, as shown in Fig. 6.10. It consists of an automatic controlled pump with a small sump for storage. Pump outlets are costly and require technicality. Possibility of deepening the outlet drain should, therefore, be investigated. The cost of installing and maintaining a pump outlet should be compared with that of excavating and maintaining a deeper outlet drain, before making a final selection.

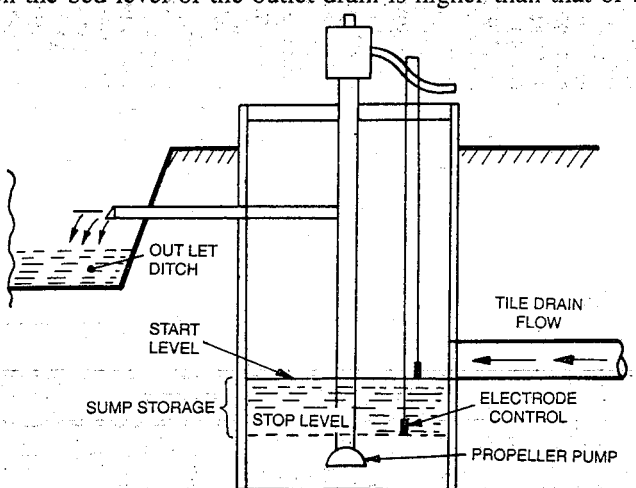


Fig. 6.10. Pump outlet (outfall) for tile drain.

6.6.3. Drawdown Curve or Movement of Water into the Tile Drains. In a fully saturated soil, water flows into the tile drain along the path shown in Fig. 6.11 (*a*). Since the quantity of water moving between any two flow lines is the same, the drawdown will be more near the tile than at the points farther away. After the saturated soil has drained for a day or so, the resulting watertable will be, as shown in Fig. 6.11 (*a*). With series of tile drains, the sub-soil water level directly over the drains, is lower than the level midway between them as shown in Fig. 6.11 (*b*).

When a filter is provided around the tile drains to surround the drains with more pervious soil, then the overall drawdown will be more. The rate of drop of watertable mainly depends upon the soil permeability and spacing of the drains. In this case, the water has to travel more distance horizontally than vertically before it reaches the drain, the horizontal permeability of the soil is more important. The permeabilities of most of the soils decrease with depth. This change in permeability affects the shape of the flow lines and the rate of the fall of watertable.

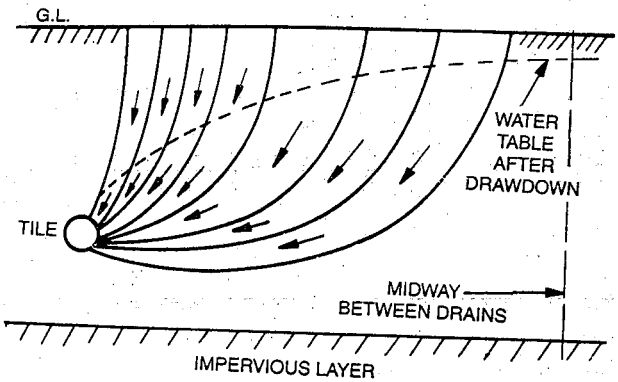


Fig. 6.11. (*a*) Drawdown curve with a single tile drain.

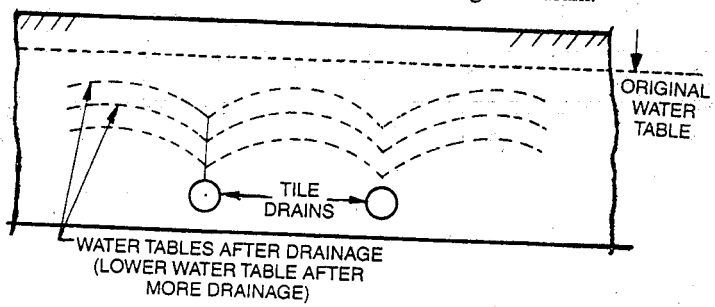


Fig. 6.11. (*b*) Drawdown curve with a series of tile drains.

6.6.4. Depth and Spacing of the Tile Drains. The closed drains are generally spaced at such a distance as to be capable of lowering the watertable sufficiently below the root zone of the plants. For most of the plants, the top point of the watertable must be at least 1.0 to 1.5 metres below the ground level; although this distance may vary from 0.7 to 2.5 m., depending upon the soil and the crop.

The tile drains may be placed at about 0.3 metre below the desired highest level of the watertable. A fair idea of the spacing between the tile drains can be obtained based on the above theory, as follows :

Let S be spacing between the drains, and a be the depth of impervious stratum from the centre of the drains, as shown in Fig. 6.12. Let the maximum height of the drained

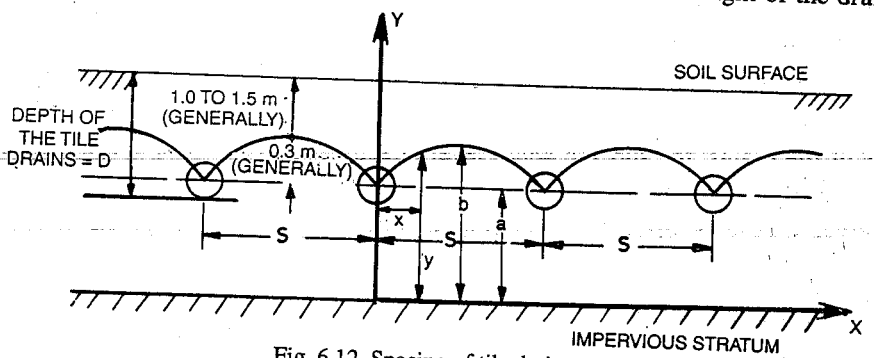


Fig. 6.12. Spacing of tile drains.

watertable above the impervious layer be b. At any distance x from the centre of a drain, let the height of the watertable above the impervious stratum be y. Then, according to Darcy's Law, we have

$$Q = KI \cdot A$$

where K = permeability coefficient in m/sec.

$\therefore$ Discharge per unit length of the drain passing the section at y (q_y) is given as :

$$q_y = K \cdot \frac{dy}{ds} \cdot y$$

Assuming the inclination of the water surface to be small, such that the tangent $\left(i.e.\ \frac{dy}{dx}\right)$ can be used in place of sine $\left(i.e.\ \frac{dy}{ds}\right)$ for the hydraulic gradient, we get

$$q_y = K \cdot \frac{dy}{dx} \cdot y \qquad ...(6.5)$$

But when $x = \frac{S}{2}$, $q_y = 0$.

and, when $x = 0$, $q_y = \frac{q}{2}$

where q *is the total discharge per unit length carried by the drain*, so that $\frac{1}{2}q$ enters the drain from either side.

Also assuming that q is inversely proportional to the distance from the drain, we can write

$$q_y = \frac{1}{2}q - \frac{1}{2}q\frac{x}{S/2} = \frac{1}{2}q\left(1 - \frac{x}{S/2}\right)$$

or $$= \frac{q}{2S}\left[S - 2x\right] \qquad ...(6.6)$$

Equating Eqs. (6.6) and (6.5), we get

$$\frac{q}{2S}(S - 2x) = K y \cdot \frac{dy}{dx}$$

Rearranging and integrating, we get

$$\int \frac{q}{2SK}(S - 2x)\, dx = \int y\, dy.$$

Assuming the soil permeability to be constant, we get

$$\frac{q}{2K.S}\left[Sx - \frac{2x^2}{2}\right] = \frac{y^2}{2} + C \qquad ...(6.7)$$

when $x = 0$, $y = a$

$\therefore$ $$\frac{q}{2KS}[0] = \frac{a^2}{2} + C \quad \text{or} \quad C = -\frac{a^2}{2}$$

Substituting $C = -\frac{a^2}{2}$, equation (6.7) becomes

$$\frac{q}{2KS}\left[Sx - x^2\right] = \frac{y^2}{2} - \frac{a^2}{2}$$

or $$\frac{q}{2KS} x\,(S - x) = \frac{y^2 - a^2}{2}$$

or $$q = \frac{KS\,(y^2 - a^2)}{(S - x)\,x} \qquad ...(6.8)$$

Also, when $x = \frac{S}{2}$, $y = b$, equation (6.8) then becomes

$$q = \frac{KS \cdot (b^2 - a^2)}{\left(S - \frac{S}{2}\right)\frac{S}{2}} \qquad \text{or} \qquad q = \frac{KS\,(b^2 - a^2)}{\frac{S^2}{4}}$$

or $$\boxed{\mathbf{q = \frac{4K}{S}\,(b^2 - a^2)}} \qquad ...(6.9)$$

or $$S = \frac{4K}{q}\,(b^2 - a^2) \qquad ...(6.10)$$

Equation (6.10) can be used to predict the spacing (S) between the drains, if q is known. q will depend on the infiltration discharge into the ground, which should be removed by the drains. Different values have been suggested. Generally, a value equal to 1% of the average annual rainfall of a place is considered to be drained by the tile drains in 24 hours. If the average annual rainfall of the place is $P_{A.A}$ (metres), then

$$q = \left(\frac{0.01 \times P_{A.A}}{24 \times 3600}\right)(S \times 1) \text{ cumecs/m length of drains}$$

$$= \frac{0.01 \times P_{A.A} \cdot S}{24 \times 3600} = \frac{P_{A.A.} \cdot S}{86{,}40{,}000} = \frac{P_{A.A} \cdot S}{8.64 \times 10^6} \qquad ...(6.11)$$

Equating with equation (6.9), we get

$$q = \frac{4K \cdot (b^2 - a^2)}{S} = \frac{P_{A.A} \cdot S}{8.64 \times 10^6}$$

or $$S = \sqrt{\frac{(8.64 \times 10^6)\,4K \cdot (b^2 - a^2)}{P_{A.A}}} \qquad ...(6.12)$$

Hence, spacing (S) can be determined easily by using eq. (6.12).

Example 6.2. *In a tile drainage system, the drains are laid with their centres 1.5 m below the ground level. The impervious layer is 9.0 m below the ground level and the average annual rainfall in the area is 80 cm. If 1% of the annual rainfall is to be drained in 24 hours to keep the highest position of the watertable to 1 metre below ground level, determine the spacing of the drain pipes. Coefficient of permeability may be taken as 0.001 cm/sec.*

Solution. Although eqn. (6.12) can be directly used in this question, since that eqn. has been derived for designing the drains to take 1% of the average annual rainfall in 24 hours, which tallies with the given data, yet it would be prudent to use the basic eqn. (6.10) for determining the spacing of tile drains, and separately compute q, as:

$$q = \frac{\frac{1}{100} \times \left(\frac{80}{100}\right) \times (S \times 1)}{(24 \times 60 \times 60)} \text{ m}^3/\text{s}$$

$$= \frac{0.8S}{8.64 \times 10^6} \text{ m}^3/\text{s/m length of tile drain}$$

Using eqn. (6.10), we have

$$S = \frac{4K}{q}(b^2 - a^2)$$

where b = ht. of W.T. above the impervious layer
$= 9 \text{ m} - 1 \text{ m} = 8 \text{ m}$

a = depth of impervious stratum below the centre of the drains $= 9 - 1.5 = 7.5$ m

$$K = 0.001 \text{ cm/s} = \frac{0.001}{100} \text{ m/s}$$

Substituting values, we get

$$S \cdot q = 4 \times \frac{0.001}{100}(8^2 - 7.5^2)$$

or $$S \times \left(\frac{0.8S}{8.64 \times 10^6}\right) = \frac{4 \times 0.001}{100}(8^2 - 7.5^2)$$

or $$0.8S^2 = \frac{4 \times 0.001}{100} \times (8.64 \times 10^6)(8^2 - 7.5^2) = 2678.4$$

or $$S = \sqrt{\frac{2678.4}{0.8}} = \sqrt{3348} = \mathbf{57.86 \text{ m}} \quad \textbf{Ans.}$$

6.6.5. Drainage Coefficient (D.C.). The rate at which the water is removed by a drain is called the *drainage coefficient.* It is expressed as the depth of water in cm or metres, to be removed in 24 hours from the drainage area. The drainage coefficient largely depends upon the rainfall but varies with the type of soil, type of crop, and degree of surface drainage, etc. Its recommended value is 1% of the average annual rainfall to be removed per day.

In irrigated areas, the discharge through the tiles may vary between 10 to 50% of the total water applied. Since the entire area is not irrigated at the same time, the drainage area to be used to calculate tile flow is not the same as the entire tiled area, but is estimated from the area irrigated. A suitable value of drainage coefficient (DC) may be taken for the calculations, depending upon the local recommendations. Values of 1 to 2.5 cm/day for mineral soils and 1.25 to 10 cm/day for organic soils for different crops, have been suggested for humid regions, by U.S. Soil Conservation Service.

Example 6.3. *A tile drainage system draining 12 hectares, flows at a design capacity for two days, following a storm. If the system is designed using a D.C. of 1.25 cm, how many cubic metres of water will be removed during this period ?*

Solution. D.C. of 1.25 cm means that 1.25 cm of water depth from the drainage area shall be removed by the drain in 24 hours.

$\therefore$ Volume of water entering the drain per day

$$= \left(\frac{1.25}{100} \text{ m/day}\right) \times (12 \times 10^4 \text{ m}^2) = 1{,}500 \text{ m}^3/\text{day}$$

Volume of water passing the drain within 2 days of flow

$$= 2 \times 1500 = \mathbf{3000\ m^3} \quad \textbf{Ans.}$$

6.6.6. Drainage Area. The area actually drained by the tile drain system is called its *drainage area*. Sometimes, the surface water is also to be removed by the tiles. In that case, the watershed area will be the drainage area, even though it may not be entirely tiled.

6.6.7. Size of the Tile Drains. The tile drains are designed according the Manning's formula to carry a certain discharge decided by D.C. and drainage area. The drains are laid on a certain longitudinal slope varying from 0.05 to 3%. A desirable minimum working grade is 0.2%. Where sufficient slope is not available, the grade may be reduced to 0.1%. Depending upon the available slope of the soil surface and the depth of the outlet, suitable value of longitudinal slope can be given to the tiles. Their sizes can be easily evaluated from Manning's formula. 10 to 15 cm tiles are minimum recommended sizes. The minimum size for perforated tubing or pipes can be reduced, as in that case, the misalignment at joints or cracks is not a problem.

Example 6.4. *Determine the size of a tile at the outlet of a 6 hectare drainage system, if the D.C. is 1 cm and the tile grade is 0.3%. Assume the rugosity coefficient for the tile drain material as 0.011.*

Solution. 1 cm D.C. means that 1 cm of water from an area of 6 hectares is entering the tiles per day.

$\therefore$ Volume of water passing the drain in 1 day $= \left(\frac{1}{100} \times 6 \times 10^4\right) = 600 \text{ m}^3/\text{day}$

$\therefore$ Volume of water passing the drain in 1 second $= \left(\frac{600}{24 \times 3600}\right) = \frac{1}{144} \text{ m}^3/\text{s}$

$\therefore \quad Q = \frac{1}{144} \text{ m}^3/\text{s} \qquad$ Now, $\quad Q = \frac{1}{n} \cdot A.R^{2/3} \cdot S^{1/2}$

For a circular drain of diameter D, we have

$$A = \frac{\pi D^2}{4},\ P = \pi D,\ R = \frac{D}{4}$$

or $$\frac{1}{144} = \frac{1}{0.011} \times \left(\frac{\pi D^2}{4}\right) \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \left(\frac{0.3}{100}\right)^{1/2}$$

or $$\frac{1}{144} \times \frac{0.011 \times 4}{\pi} = \frac{D^2 \cdot D^{2/3}}{(4)^{2/3}} \times \frac{1}{\sqrt{333.3}}$$

or $$\frac{0.011 \times 4 \times 2.52 \times 18.26}{144 \times \pi} = D^{8/3}$$

or $$D = (0.00447)^{3/8} = 0.132 \text{ metre} = 13.2 \text{ cm} \quad \textbf{Use 15 cm dia. pipe.} \quad \textbf{Ans.}$$

Example 6.5. *Sugarcane (root zone depth 1.8 m) is grown in a particular area where the ground water table is 2.0 m below ground. If the size of the soil pores is 0.08 mm in diameter and surface tension = 0.054 N/m, is the field water-logged ? If so, determine the vertical location of closed drains below ground, spaced at 15 m. Take drainage coefficient as 0.116 cumecs/km^2. Coefficient of permeability as 10^{-6} m/s, and the impervious stratum to occur at 7.0 m, below ground.*

Solution. We have stated in article 6.4 that if the water table rises up or if the plant roots happen to come within the capillary fringe, the salinity and water-logging of the land occurs. Here we have to see as to whether the plant roots (extending 1.8 m below the ground level) do come within the water-table or extended water table (due to capillary fringe). The actual water table is 2 m below ground and hence is not directly affecting the roots. But the capillary fringe is to be seen.

The height to which water rises by capillary action in a soil is given by eqn. (6.3) in "Soil Mechanics and Foundation Engineering" by the same author, as

$$H_e = \frac{4T \cdot \cos \alpha}{d \cdot \gamma_w}$$

where $\alpha = 0$ for max. capillary height

d = Dia of pores = 0.08 mm = 0.08×10^{-3} m

γ_w = Unit wt. of water = 9.81×10^3 N/m^3

T = Surface tension = 0.054 N/m

$$\therefore \quad H_e = \frac{4 \times 0.054 \times 1}{0.08 \times 10^{-3} \times (9.81 \times 10^3)}$$

$$= \frac{4 \times 0.054}{0.08 \times 9.81} = 0.28 \text{ m}$$

The water-table thus effectively stands at 2 – 0.28 = 1.72 m below the ground, thereby causing slight water logging, since the roots extend up to 1.8 m below the ground. **Ans.**

In order to design the tile drains, we represent the given conditions in Fig. 6.13 as follows. Let the tile drains be laid at a height 'a' above the top of the impervious layer.

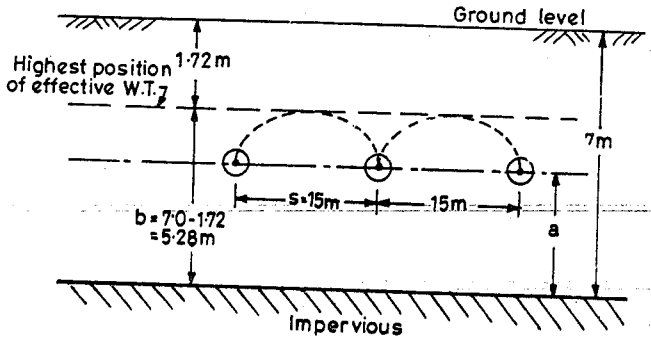


Fig. 6.13.

From eqn. 6.9, we have

$$q = \frac{4K}{S}(b^2 - a^2) \qquad ...(i)$$

where $S = 15$ m (given)

$K = 10^{-6}$ m/s (given)

$b = 7 - 1.72 = 5.28$ m

$a = ?$

q = discharge carried by each tile drain in cumecs/m run

The tiles have a drainage coefficient of 0.116 cumecs /km^2. Each tile carries discharge from 15 m width and 1 m length (considering 1 m length $\perp$ to diagram).

$\therefore$ The discharge carried by each tile drain of 1 m length, q is given as:

$$q = 0.116 \frac{\text{Cumecs}}{\text{km}^2} \times \text{Area in km}^2$$

$$= 0.116 \times \left(\frac{15}{1000} \times \frac{1}{1000}\right) \text{cumces} = 1.74 \times 10^{-6} \text{ cumces/m run}$$

Substituting values, in (i) we get

$$1.74 \times 10^{-6} = \frac{4 \times 10^{-6}}{15}(5.28^2 - a^2)$$

or $a = \mathbf{4.62\ m}$

The tile drains should therefore be laid at $7 - 4.62 = \mathbf{2.38\ m}$ below the ground. **Ans.**

6.6.8. Materials of Tile Drains. Pipes used for tile drains are generally made from materials, such as clay or concrete, in short lengths. Sometimes, they may be made of bituminous fibre or steel. Corrugated plastic perforated pipes are gaining popularity, because of its lightness and reduced labour in handling it.

Comparison between clay and concrete pipes. Good quality concrete pipes are very resistant to freezing and thawing, but may deteriorate in alkaline and acidic soils. Clay pipes, on the other hand, are not affected by acid or alkaline soils. When subjected to continuous freezing or thawing conditions, concrete pipes are found to be safer than the clay pipes, although clay pipes are resistant to frost damage. Both the types of pipes should have sufficient strength, so as to withstand static and impact loads transmitted from the soil above.

Good clay or concrete pipes should have the following characteristics:

(*i*) Resistant to weathering and deterioration in the soil.

(*ii*) Low water absorption, *i.e.* high density.

(*iii*) Uniformity in shape and wall thickness, etc.

(*iv*) Freedom from defects, such as cracks, etc.

(*v*) Resistant to freezing, thawing and frost damage.

(*vi*) Sufficient strength to withstand static and impact loads for which designed.

6.6.9. Layout of Tile Drains. The tile drains may be aligned in different fashions, depending upon the topography of the area. Generally, laterals (branch drains) run through most of the drainage area and join the mains, which in turn, outfall into some deep open drain. The depth of the deepest tile drain shall be kept within the range of 3

metres from the surface, since the root zones of crops get affected by water within this depth range. A simple network of the drainage arrangement is shown in Fig. 6.14.

Various possible alternative **layouts for the tile drainage system** are shown in Fig. 6.15, and are discussed below:

(1) **Natural system.** The natural system is generally adopted *in rolling topography, where drainage of isolated areas is required.* The mains and the connected laterals are provided in natural course, as shown in Fig 6.15 (*a*). This system is suitable when the land is not to be completely drained. The system is quite flexible and permits location of drains where they are most needed.

(2) **Grid iron system.** This drainage system, consisting of laterals and mains (or submains), is shown in Fig 6.15 (*b*). In this

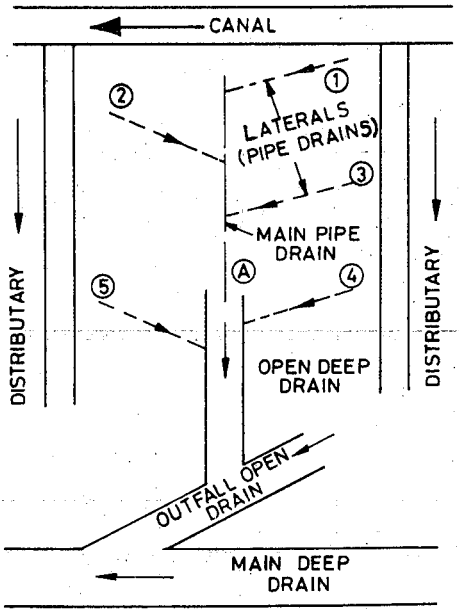
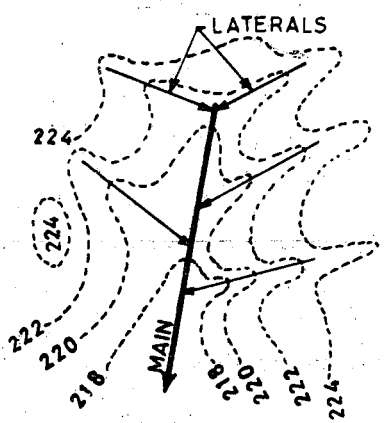
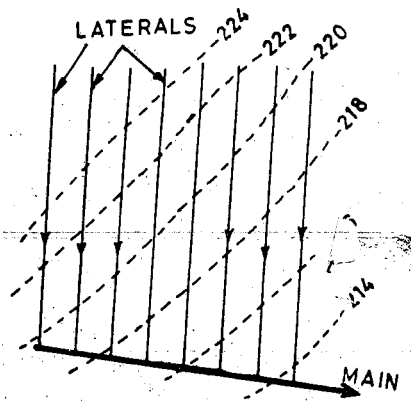


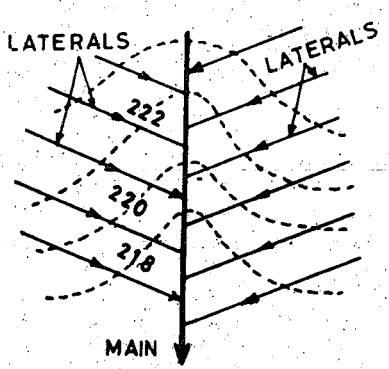
Fig. 6.14. General layout of a tile drain network.



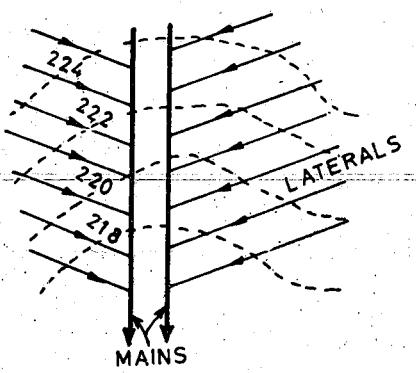
(*a*) Natural System



(*b*) Grid Iron System.

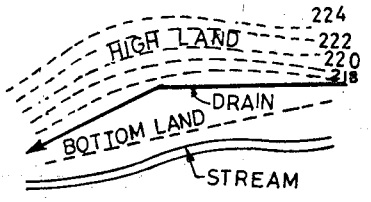


(*c*) Herring Bone System



(*d*) Double Main System.

(Fig. 6.15 continues)



(*e*) Intercepting Tile Drains System.
Fig. 6.15. Different Layouts of Tile Drainage Systems.

system, the laterals are provided only on one side of the main, as shown. *This system is adopted when the land is practically level; or where the land surface slopes away from the sub main on one side, and when the entire area has to be drained.*

(3) **Herring bone system.** In this layout pattern, laterals join the mains (or sub-mains) from each side, alternatively, as shown in Fig 6.15(*c*). This layout is adopted, when the main (or sub-main) is laid in depression. *The land along the main is double drained, but since it exists in depression, it probably requires more drainage than the land on the adjacent slopes.*

(4) **Double main system.** This system has two mains with separate laterals for each main, as shown in Fig 6.15(*d*). *This layout is adopted when the bottom of depression is wide.* This arrangement reduces the length of the laterals and eliminates the break in slope of the lateral at the edge of the depression.

(5) **Intercepting tile drains system.** In this system, there is no lateral drain. A main (or submain) is provided at the toe of the slope, as shown in Fig 6.15(*e*). *This arrangement is preferred when the main source of drainage is from a hilly land.*

In general, all mains and submains should be kept away from trees, as the roots of trees can easily enter the open joints of the tile drains, and thereby blocking them.

PROBLEMS

1. (*a*) What is meant by "water-logging"?

(*b*) What are the principal causes and effects of water-logging in a canal irrigated farm?

(*c*) What precautions and measures will you adopt to prevent water logging of irrigated lands?

2. (*a*) What are the principal causes of water-logging in a canal irrigated tract?

(*b*) What precautions will you observe in constructing a new canal system to avoid water-logging?

[**Hint.** Provide lined canals equipped with under-drainage system]

(*c*) What steps will you take to improve an already water-logged tract?

3. What is meant by water-logging? What are its ill effects? Describe some anti-water-logging measures with suitable sketches.

4. Discuss different types of drainage systems provided in irrigated tracts as precautions against water-logging. Give salient features of an open drain system.

5. (*a*) What is meant by "saline" and "alkaline" soils?

(*b*) What precautions will you adopt to prevent salinity of irrigated land?

(*c*) How will you proceed to reclaim saline land?

6. (*a*) Discuss critically the statement : "intensive irrigation leads to reduced crop yields".

(*b*) What are "tile drains" and how do they help in preventing water logging? How will you decide the depth and spacing of tile drains?

7. (*a*) What is meant by "Land Drainage", and where is it needed?

(*b*) What are the two principal types of drainage systems necessary for draining irrigated tracts to avoid their water-logging?

8. (*a*) What is meant by "open drainage" and "closed drainage" as applied to draining irrigated tracts?

(*b*) A tile drainage system draining 15 hectares, flows at a design capacity for three days, following a storm. If the system is designed using a drainage coefficient of 2.25 cm, how much water will be removed during this period?

[**Hint.** Follow example 6.3] [**Ans.** 10125 m^3]

9. (*a*) What is meant by "Drainage coefficient" and how does it help in determining the sizes of tile drains to be laid for preventing water-logging of irrigated farms?

(*b*) What are the different materials which are commonly used for the drains, and what are their comparative merits and demerits?

(*c*) Determine the size of a circular tile drain, draining 6 hectares of a drainage area, if the drainage coefficient (D.C.) is 1.5 cm and the tile grade is 0.4%. Assume the rugosity coefficient for the tile material as 0.013.

[**Hint.** Follow example 6.4]

10. What is meant by "tile drains" and what are the different methods of aligning them? How will you proceed for fixing their depth, spacing and sizes? What are their advantages?

11. Write notes on any two of the following:

(*i*) Open drainage (*ii*) Tile drainage

(*iii*) Surface inlet (*iv*) Blind inlet or French drain

(*v*) Envelope filters (*vi*) Gravity outlets

(*vii*) Pump outlets (*viii*) Movement of water into tile drains

(*ix*) Random and Herring bone types of tile drainage systems.

12. Fill in the blanks with appropriate words:

(*i*) If the soil is more permeable, the chances of water-logging are ..... (less/more)

(*ii*) Intensive irrigation should be ..... in areas susceptible to water-logging. (adopted/avoided)

(*iii*) Extensive irrigation should be ..... in areas susceptible to water-logging. (adopted/avoided)

(*iv*) ..... irrigation helps in checking water-logging. (Flow/Lift)

(*v*) The remedial measures for water-logging are .....

(*vi*) Drainage schemes are implemented for ..... water-logged areas.

Ans. (*i*) more; (*ii*) avoided; (*iii*) adopted (*iv*) lift; (*v*) installing lift irrigation schemes, and implementing drainage schemes; (*iv*) reclaiming.]

7

Hydrology and Run off Computations for the Design of Hydraulic Structures across Rivers and Streams

INTRODUCTION TO HYDROLOGY

7.1. Definition, History and Importance of Hydrology

Hydrology may be defined as the science that deals with the depletion and replenishment of our water resources. It deals with surface as well as ground waters, in as far as their occurrence, circulation, distribution, chemical and physical properties, reactions to environment and living beings, etc., are concerned. In fact, hydrology embraces the entire domain of water, its history and quantitative use on the earth.

Hydrology is comparatively a new science. Even in the pre-historic periods, it seems to have remained under-developed, although many other sciences, such as mathematics, astronomy etc., were fully developed. This science, perhaps, found its origin in the year 1930, and since that time, it has progressed considerably, giving us more and more confidence in predicting hydrological events, such as rainfall, runoff, ground water accretion, etc.

The basic knowledge of this science is a must for every civil engineer, particularly the one who is engaged in the design, planning or construction of irrigation structures, bridges and highway culverts, or flood control works, etc. Some of the various questions, which this science will be called upon to answer, while designing the above engineering feats, are :

(*i*) What maximum flows, after different years are expected at a spillway, or a highway culvert in a city drainage system ?

(*ii*) What minimum reservoir capacity will be sufficient to assure adequate water for irrigation or municipal water supply, during droughts ?

(*iii*) How much quantity of water will become available at a reservoir site, and when will it become available ? In other words, what will be the available possible supplies at the reservoir site ?

(*iv*) What effects will be there on the river water levels after the construction of proposed reservoirs, levees, or any other controlling structures ?

Generally, specialist hydrologists are rarely employed for answering such questions; and the civil engineers responsible for the planning, design and construction of hydraulic structures, are themselves required to answer them ; thereby making it imperative upon them to gain a thorough knowledge of Hydrology. Only then, they will be able to design efficiently and economically functioning hydraulic structures.

In the following pages of this chapter, we shall discuss the various hydrologic principles involved in this science, as well as, its applications to engineering problems.

7.2. The Hydrologic Cycle

Most of the earth's water sources, such as, rivers, lakes, oceans and underground sources, get their supplies from the rains, while the rain water in itself, is the evaporation

from these sources. Water is lost to the atmosphere as vapour from the earth, which is then precipitated back in the form of rain, snow, hail, dew, sleet or frost, etc. This evaporation and precipitation continues for ever, and thereby, a balance is maintained between the two. This process is known as *hydrologic cycle*. It can be represented graphically as shown in Fig. 7.1.

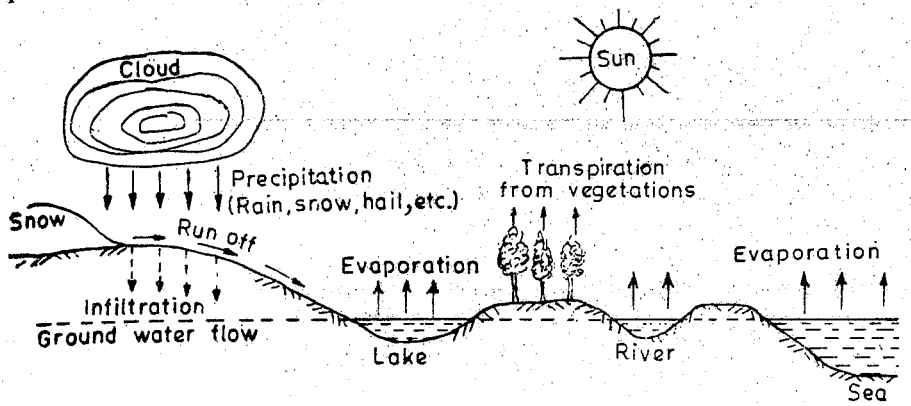


Fig. 7.1. Hydrologic cycle.

Sun and Coriolis force play important parts in completion of hydrologic cycle. *Sun* evaporates water, and *Coriolis force,* by causing and controlling winds, circulates the evaporated water vapour, and thus, helping in its reprecipitation at different places.

WEATHER AND ITS PRECIPITATION POTENTIAL

7.3. Definition of Precipitation

Water evaporates from exposed water surfaces like streams, rivers, oceans, ponds, and also from the lands and plants, in the form of water vapour. These water vapour get collected in the atmosphere and behave like a gas. Under a normal range of temperature and pressure, the water vapour obeys the various gas laws (*i.e.* Boyle's law, Charle's law, etc.). As the evaporation continues, the amount of atmospheric vapour goes on increasing. But since a space (atmosphere) can hold only a certain fixed amount of water vapour in the presence of a solid or a liquid surface, a state is reached when any further addition of vapour will get condensed on the surfaces. The vapour may get condensed in different forms, such as mist, rain, hail, snow, sleet etc. The evaporated water thus returns to the earth surface in any of these forms. *The water which comes back to the surface of the earth in its various forms, like rain, snow, hail, sleet, etc. is known as precipitation.* A major part of the precipitation occurs in the form of rain, and minor part occurs in the form of snow. Other forms of precipitation, such as *hail, sleet, mist,* etc., are negligibly small, and generally ignored in the design of most of the hydrological works; and, therefore, they are not of much importance to us.

7.4. Saturation Pressure

As stated above, water vapour are generally present in the atmosphere. Sometimes, they are alone and sometimes they are mixed with other gases. The pressure exerted by each component of such a mixture is known as *partial pressure.* The pressure exerted by the vapour present in the air is known as *vapour pressure,* and if the air is fully saturated with these vapour, it is then known as *saturation vapour pressure or* **saturation pressure.**

If the barometric pressure is kept constant and the temperature is reduced; *i.e.* if the air is cooled at constant barometric pressure, a stage will come when the air will become saturated with the same amount of vapour. This temperature is known as **dew point.** If the cooling is continued, the vapour will get condensed on the contact surfaces. This condensation will be in the form of *dew,* if the dew point is greater then 0°C; and it will be in the form of *frost* if the dew point is less than 0°C.

7.5. Lapse Rate

In order to understand climate and rain-formation, it is necessary to understand the changes of temperature with height, in the atmosphere. In fact, not only temperature, but also density and pressure of the atmosphere go on decreasing, as the height above the surface of Earth (altitude) increases, as shown in Fig. 7.2.

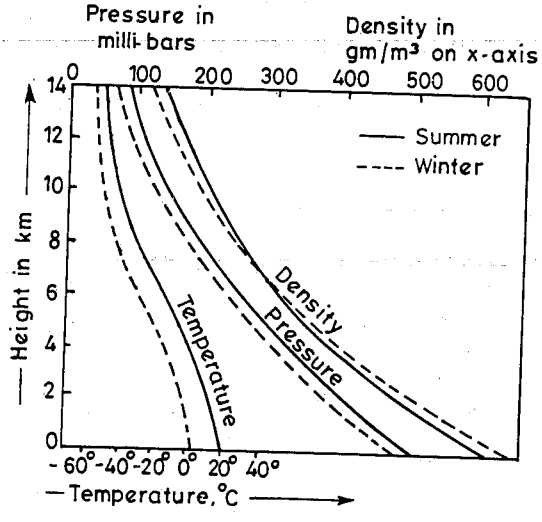


Fig. 7.2. Variation of temperature, pressure and density of atmosphere with altitude.

The decrease in temperature of the undisturbed free atmospheric air, with altitude, is called *Lapse rate* or *Temperature lapse rate.* The general variation of temperature with altitude, through different zones of atmosphere (such as *troposphere, stratosphere, ionosphere*) is shown in Fig. 7.3

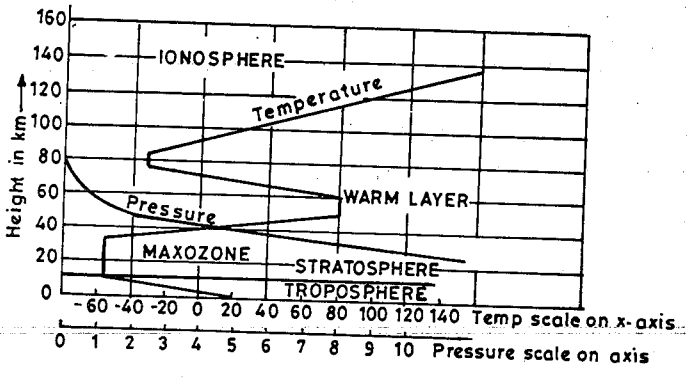


Fig. 7.3. Variation of temperature and pressure through different zones of atmosphere below the exosphere.

7.5.1. Dry Adiabatic Lapse Rate. When air expands, its given volume will become rarer and lighter ; and when it contracts its given volume will become dense and heavier. The process of expansion, as the light air rises ; and the compression as the heavier air sinks, involves no transfer of heat to or from the surrounding air, and is hence

called an *adiabatic process.* Although, there is no loss of heat, yet there is a definite change in temperature, solely due to the process of expansion and contraction. Expanding air will *drop* its temperature at the rate of 10°C for every km of ascent. This *fall* in temperature is much more than the lapse rate of free air under normal condition (*i.e.* 6.5° per km) ; and is called the *dry adiabatic lapse rate.* This is shown in Fig. 7.4. The temperature rise in compression is equivalent to the temperature fall in expansion.

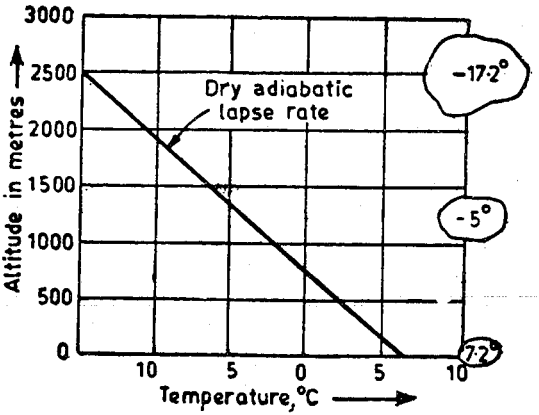


Fig. 7.4. Dry adiabatic lapse rate.

As air is cooled adiabatically, it eventually reaches in altitude where its decreased temperature corresponds to the 'saturation pressure; in other words, the temperature cannot fall further without condensing some of its water vapour. The altitude where this condensation begins, is called the **condensation level,** and the corresponding temperature as the *dew point.*

7.5.2. Adiabatic Saturation Lapse Rate. As the air continues to expand, the process of condensation also continues, and the fall in temperature becomes less rapid. This temperature change is now called *adiabatic saturation lapse rate.* This rate of change of temperature during the moist adiabatic process (when condensation is taking place) ranges from 3 to 10°C per km of altitude rise, being smaller for warm saturated air and larger for cold air.

7.6. Humidity and Relative Humidity

The term humidity is used in order to obtain an idea of the amount of moisture present in the air. The amount of moisture present in the air, when expressed as mass per unit of volume, is known as *Absolute humidity.*

Absolute humidity at a given temperature

= Mass of moisture present in a unit volume of air at the given temperature.

Relative Humidity is defined as the ratio of the actual vapour pressure to the saturation vapour pressure at the same temperature. Hence, it gives an idea of the extent to which the air is saturated.

Relative humidity (R.H.)

$$= \frac{\text{Actual vapour pressure at a given temperature } (e_a)}{\text{Saturation vapour pressure at the same temperature } (e_s)} \qquad ...(7.1)$$

Sometimes, relative humidity is defined as the mass per unit volume of the actual vapour present in the air, to that it could contain at the same temperature when fully saturated.

Humidity can be measured either by a *psychrometer* or by a *hygrometer. Absolute humidity* can also be measured by directly passing a known volume of air through a drying agent, which absorbs the moisture of air. The increase in mass of this drying agent divided by the volume of the air passed, will give the absolute humidity of the air.

Absolute humidity decreases rapidly, as we go at higher and higher altitudes. About one-half of the total moisture present in the atmosphere occurs only within a distance

of one mile from the earth's surface. Humidity decreases with altitude, because at higher altitude, the convective currents which are responsible for carrying the water vapour into the air, are very much reduced.

7.7. Weather and its Role in Causing Precipitation

The prospective precipitation in an area is primarily dependent upon the *weather conditions* of the area. The weather, inturn, depends upon the *atmospheric pressure, temperature, humidity (moisture),* etc. The interaction between these variables, infact leads to formation of *winds* and *cyclones,* which cause precipitation.

The branch of science, which exclusively deals with the study of weather, is known as meteorology, and, hence, a hydrologist, or a water resources expert, is to a larger extent, dependent upon a meteorologist, because all rainfall predictions and *measurements* are done by a meteorologist.

Even in our country, *Indian Meteorological Department* (IMD) is responsible for all weather and rainfall predictions, and this Department controls the measurement of rainfall and snowfall, although most of the *rain gauges* in our country are maintained by the State Governments, but under the technical control of IMD. In the National Capital Territory of Delhi, however, IMD itself is maintaining the rain gauges.

7.7.1. Formation of Clouds. We have stated earlier that the condensation of water vapour in the atmosphere, as caused by the cooling of the air mass, causes precipitation. Infact, this is a simplified statement, because such condensation does not necessarily cause precipitation, but it certainly forms **clouds** or **fogs.** *It is the formation of clouds, which ultimately causes precipitation.*

A cloud is, thus, formed by the cooling and condensation of the rising vapour (air) into some visible aggregates of minute particles. *A cloud, therefore, generally contains ice particles and small droplets of water.* The nature of the cloud formed, largely depends upon the type of the ascending motion, to which the air gets subjected to, as discussed below :

(*i*) *Prolonged wide spread slow ascent of air,* as in a low pressure area, or *stirring motion,* gives rise to stratification in clouds. Such clouds are formed in layers, and may be called **stratiform clouds.** Such clouds lead to *cyclonic precipitation.*

(*ii*) *Convective or violent ascent of larger bubbles of air,* resulting from **insolution** (*i.e.* cold air coming over a warm region), may lead to formation of *heap* type of clouds, called **cumuliform clouds.** Such clouds lead to *convective precipitation.*

(*iii*) When air is forced to ascend a hill or a mountain barrier, typical **orographic clouds** are formed, which lead to *orographic precipitation.*

Conventional classification of clouds

Depending upon the elevation at which the clouds are formed, they may be classified as :

(*i*) *high clouds ;*

(*ii*) *medium clouds ;*

(*iii*) *low clouds; and*

(*iv*) *vertically developing clouds.*

The levels at which these clouds form, may differ in *equatorial regions (tropics), temperate latitude regions,* and *polar regions* to some extent, as shown in Table 7.1

Table 7.1. Formation of Clouds at Different Levels

Sl. No.	*Type of cloud (basic group)*	*Level of occurrence in different regions*		
		Tropical regions	*Middle latitude regions*	*Polar regions*
(*i*)	High clouds	6–18 km	5–12 km	3–8 km
(*ii*)	Medium clouds	2–8 km	2–7 km	2–4 km
(*iii*)	Low clouds	0–2 km	0–2 km	0–2km
(*iv*)	Vertically developing clouds	(*a*) base 0.5–2 km (*b*) tops* 18–20 km	0.5–2 km 10–12 km	0.5–2 km 5–8 km

7.7.2. Basic Forms of Clouds. There are three basic forms of clouds: (*i*) *stratiform* (**stratus**) ; (*ii*) *cumuliform* (**cumulus**) ; and (*iii*) *fibrous* (**cirrus**). Despite the infinite variety of forms, it is possible to classify clouds into the *basic groups* or *genera.* These can further be sub divided into *species* according to their shape and structure.

The makes of the clouds and the distinguishing features of the genera, as given in Table 7.2, will help in their identification.

Table 7.2. Distinguishing Features of Clouds

Type of clouds	*Distinguishing features of genera*
(A) **High Clouds,**	
(*i*) *Cirrus* (Ci)	Occur in the form of detached white patches or filaments or narrow bands with fibrous silky appearance. They are composed of ice crystals at temperatures below 0°C.
(*ii*) *cirrocumulus (Cc)*	Occur in the form of thin white sheets or small patches or elements in the form of grains, ripples, etc., merged or separate, more or less regularly arranged. They are composed of ice crystals.
(*iii*) *Cirro-stratus* (Cs)	Transparent whitish cloud veil, either smooth or fibrous, often producing a halo. They are composed of ice crystals.
(B) **Medium Clouds,**	
(*iv*) *Alto-stratus* (As)	They are grey striated or fibrous veils, like Cs, but thicker, through which sun is either seen only as diffuse bright patch or not seen at all. They are mixed clouds of ice crystals and water droplets. They may give rise to corona.
(*v*) *Alto-cumulus* (Ac)	These clouds occur as white grey sheet or layer of rounded masses, rolls, etc. arranged in groups, lines or waves. They are composed predominantly of water droplets, sometimes with a few ice crystals.
(*vi*) *Nimbo-stratus* (Ns)	They are amorphous dark-grey clouds, sometimes reaching low, *rain or snow falls from such clouds*, and they are thick enough to obscure the sun. They are mixed clouds with ice crystals at top (when the top is very high) and water droplets below.
(C) **Low Clouds,**	
(*vii*) *Strato-cumulus* (Sc)	Such clouds occur as layer or patch, consisting of laminae or globules in groups, lines or waves with soft grey appearance. They are composed of water droplets at temperatures above 0°C.
(*viii*) *Stratus* (St)	They are uniform featureless layer clouds, akin to fog but not resting on ground. When broken into irregular ragged pieces, they are known as fracto-stratus. Composed of water droplets above 0°C.
(D) **Vertically developing Clouds**	
(*ix*) *Cumulus* (Cu)	They are detached dense clouds with marked vertical growth in the form of rising white domes or towers ; tops looking like cauliflowers. Base nearly horizontal. White tops when sunlit. Composed of liquid water with large droplets.
(*x*) *Cumulonimbus* (C_b)	They are heavy dense masses with large vertical growth, appearing like mountains or towers. Upper portion develops fibrous structure resembling an anvil or plume. They produce lightening, thunder, showers, hail, squall developing into a thunderstorm. They are composed of liquid droplets at bottom and ice crystals in the portion above.

* The tops reach higher in the tropics, because tropopause land is higher.

7.7.3. Measuring the Clouds. The clouds are generally measured *visually.* The size of the patch, in which the clouds occur, is estimated mentally as a part fraction of the total sky. This amount is reported in *oktas**, or eighths of the sky covered by the clouds. When observing the clouds, the amount of each type of cloud present in the sky is recorded separately, as also the total coverage of the sky by all the clouds put together. Total cloud count equal to 8 oktas, would evidently mean a fully overcast sky. The height of the base of the clouds is estimated by optical instruments.

7.7.4. Formation of Rain Drops and Snow Flakes in Clouds. Evidently, a large quantity of water always remains suspended in a cloud. This water remains present in the form of *fine droplets,* which remain floating. These droplets do have a tendency to fall down under their own weight by gravity, but this tendency is opposed by the upward motion of the atmosphere. Hence, for a droplet to fall and reach the ground as precipitation, it must grow to a sufficiently large size, so as to acquire fall speed more than the up-current.

Depending upon the size of the falling droplets from the cloud, precipitation may occur in the form of drizzle, *rain* or *hail.* The smaller diameter droplets give rise to **drizzle ;** whereas, larger diameter droplets produce **hail stones.** *Medium sized droplets of the order of 300—4000 micron** in diameter, fall down as* **rain drops :** as shown in Table 7.3. The *intense showers* may still be having larger sized droplets, say up to 10,000 microns.

Table 7.3

S. No.	*Form of Precipitation*	*Droplet size diameter in microns*	*Fall speed in m/sec*
1.	Drizzle drops	40—500	1
2.	Rain drops	500—4,000	10
3.	Hail stones	10,000—50,000	20—50

Infact, a cloud initially contains small droplets of the size of 40 microns. Somehow or the other, the droplets grow in size to fall down as precipitation. This **growth of the droplets** have been directly related to : (*i*) cloud thickness ; and (*ii*) magnitude of up-currents. *The larger the cloud thickness, and stronger are the up currents, the bigger will be the size of the droplets formed.*

The mechanism responsible for the growth of the droplets in a cloud, has been related to the two different theories, which are applicable separately to *warm clouds* and *cold clouds,* as discussed below :

(1) *Coelescence theory for warm clouds.* In tropical regions, rains occur from clouds which are not cold enough to produce ice crystals. The growth of droplets in such warm clouds is considered to be under the action of *aggregation of smaller drops.* A droplet bigger than its neighbours, sweeps and catches a large number of droplets in a cone, on its downward journey. By collision and coelescence, the drops grow to limiting size, and breakup. The larger fragments fall to the ground as rain, while the smaller ones evaporate. *This is a self-sustaining chain reaction.*

The larger drops are formed in such clouds, because of the large magnitudes of the vertical up currents in the tropics. In the tropical maritime regions, showers can therefore occur even from clouds with lesser vertical development, compared to inland areas. This is explained to be due to availability of giant nuclei (initial droplets) in the form of *large sea spray particles,* which can easily form the larger drops, to initiate the chain reaction of further coelescence.

* 1 oktas = 1/8th of full sky.

** 1 micron = 10^{-6} m = 10^{-3} mm.

This process excludes the ice-crystal phase, and explains the *heavy warm showers* of the tropics, caused by warm clouds, which do not rise up to the freezing level.

(2) *Ice crystal theory for cold clouds.* The clouds, which rise to higher levels, having temperatures of the order of –12°C, will normally contain a vast number of *super-cooled drops* at the top of the cloud, along with a small number of *ice crystals* below. Even dust particles can help in providing these initial ice crystals, called *ice nuclei.*

For such freezing temperatures, evidently the saturation vapour pressure is lower at an ice surface than at a water surface. The air in the cloud will therefore, have a vapour pressure somewhere between these two saturation pressures ; and hence, the water droplets will evaporate, and further condensation will occur on the ice particles, thereby increasing their sizes. This process leads to preferential growth of ice crystals by direct *sublimation* at the expense of super cooled water drops. This growth of ice crystals is most intense at temperatures around –12.5°C, when the difference in vapour pressure over ice surface and that over water surface, is maximum. The resultant **snow flakes,** so formed in such clouds, fall down to lower levels, and melt in the warm region below. If, however, the temperatures continue to be very low, **snowfall** occurs.

Another phenomenon, which may help in increasing the number of ice crystals in such clouds, is the **splintering process.** In this process, initially an *ice-shell* is formed on the super-cooled drop. As the ice-shell grows, and presses the drop inwards, the drop gets shattered, causing the inside water to eject out in minute jets, which immediately freeze into *spikes.* The spikes are finally splintered, scattering fine ice crystals in the cloud.

7.8. Scanning and Predicting Weather

The advent of modern machines, like RADARS, and more recently of SATELLITES, have enabled us these days to thoroughly scan the weather around the globe, and to predict the rain *storms, cylclones, winds, squall, thunderstorms,* etc., well in advance, by days and weeks.

Weather forecasts are thus issued to give advance warnings to the public in general, and *aviation authorities* in particular, regarding the possible atmospheric disturbances in their areas.

Short range forecasts are valid up to 48 hours, and are precisely worded ; whereas, **long range forecasts** are given between 1–2 weeks in advance, or over longer periods. Evidently, the **medium range forecasts** are valid for periods ranging between 2 to 7 days.

Short range forecasts, as are announced daily on our T.V. programme, cover the imminent possibilities of *atmospheric disturbances,* including information on winds, temperature, weather, clouds, thunderstorms, rainfall, snowfall, etc. They are considered to be the most important in all weather forecasts.

Long term forecasts are generally framed to give an idea of the *mean state of the atmosphere* or *trends.* Mean precipitation and temperature are the elements most commonly forecast for long periods. They are usually indicated as departures from the normal. *Seasonal rainfall forecasts,* issued in our country by IMD over the general outlook of the coming yearly monsoons, well before the commencement of the season, are covered in this type of forecasts.

7.8.1. Weather Radars. The word RADAR, as we all know, stands for *'Radio Detection and Ranging'.* In general, such machines are used in *aviation* for detecting and guiding aeroplanes, etc. besides their use in weather detection. Since the radars used in weather scanning differ from the ones used for aviation purposes, they are here called as *weather radars* or *meteorological radars.*

A weather radar, infact, is a powerful tracking device, used for locating *thunderstorms, rain-storms, duststorms, cyclones,* etc. The radar, infact, locates and keeps a watch on the movements of such atmospheric disturbances, which together are called *metors**. A radar can, thus, help in estimating the stage of development of thunderstorms, dust storms, squall, rainfalls, snowfalls, etc. It can estimate the direction and speed of the movement of severe weather phenomena, and their formation, growth, and dissipation. It can also estimate the height of the bases and tops of the clouds, and freezing level. The quantitative estimate of precipitation is also possible (for which it is seldom used).

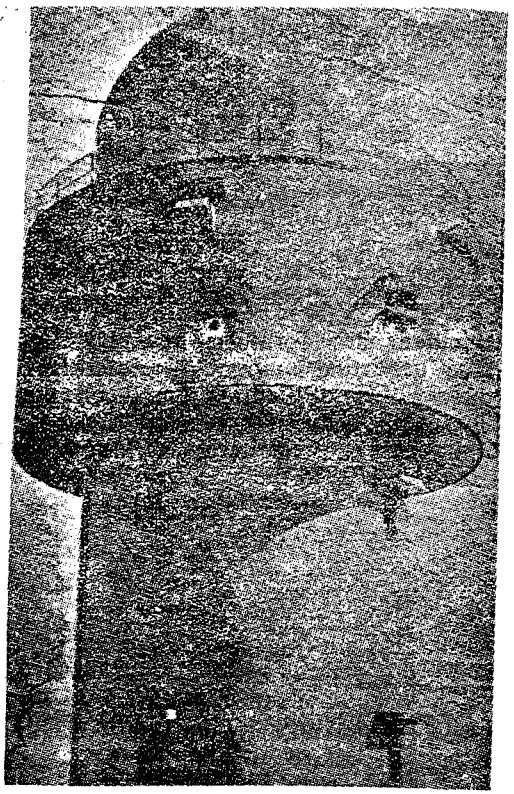


Fig. 7.5. A PHOTOVIEW of a radar, shielded by transparent dome, covers the west coast from Goa to Gulf of Cambay. It gives a bird's eye view of rain bearing clouds within an area of approximately 5,00,000 sq. km. around Bombay.

Weather radars are most widely used for locating and keeping track of cyclones, which are liable to cause wide spread misery in coastal areas. In supplementing rain gauge measurements, they are used for determining the *areal extent, orientation* and *movement of rainstorms.* Hence, in a unified rain measuring arrangement, the rain gauges are used for measuring the total amount as well as the intensity of a rainfall ; and the weather radar is used for determining the areal extent, orientation, and movement of such a rain, in addition to taking certain sample measurements of rain intensity and rainfall quantity for checking gauge readings. *A radar thus, helps in checking and supplementing rain gauge data,* in addition to its use in weather forecasting.

A radar, infact, releases a narrow beam, consisting of regular succession of pulses of electromagnetic radiation. This *radiated beam,* travels in a particular direction (being determined by the direction of the movable antenna) at the speed of light, and is partially reflected back, by clouds or precipitation particles. This reflected wave is received by the same antenna at the radar station. The electromagnetic energy of the reflected wave is lesser than the transmitted energy, as certain energy is lost in passing through the precipitation target. This reflected energy is represented on the radar screen, and is called an **echo**. The brightness and the shape of the echo formed, will indicate the type of the prevailing hydrometeor in the path of the transmitted beam.

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* **Meteors** consist of *precipitation, suspensions* or *deposits* of liquid or solid particles, optical or electrical manifestations. Depending upon the composition of particles and the physical processes involved, meteors may be classified as : *hydrometeors, lithometeors, photometeors* or *electrometeors.*

Hydrometeors include *haze, mist, fog, drizzle, rain, shower, snow, hail, dew, hoar-frost, rime* and *glaze* etc. **Lithometeors** include *smokes, dust-winds, sand storm,* etc. **Photometeors** include *mirage, glory, halo, corona, rainbow,* etc. **Electrometeors** include *lightening, thunder including thunderstorms, St. Elmos' fire, aurorae,* etc.

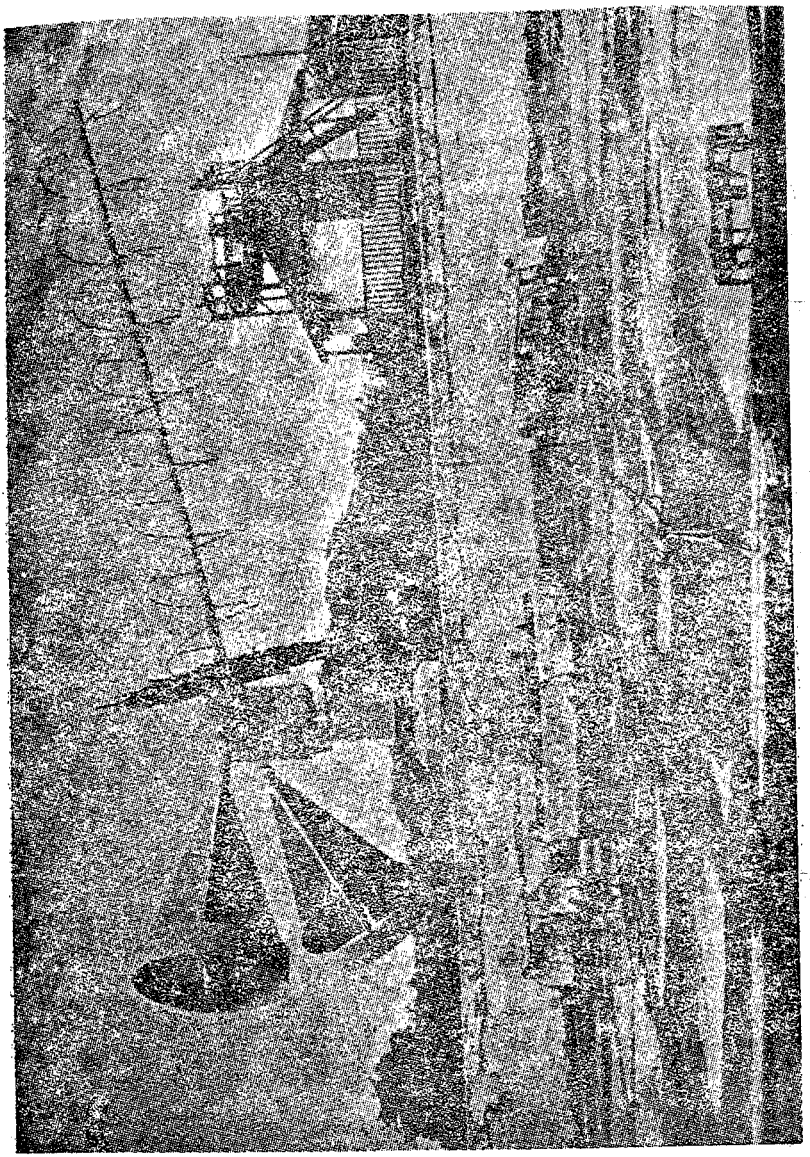


Fig. 7.6. The ANTENNA of the Automatic Picture Transmission equipment in Bombay is operated by remote control. In the background is the observatory tower at the Regional Meteorological Centre, which is about 130 years old.

Different kinds of hydrometeors give rise to different types of echoes, having distinct characteristics. Even convective and stratiform clouds produce different echoes. Different weather phenomena like thunderstorms, dust storms, line squalls, hail, tornado, snowfall, etc. display different characteristic features, such as *bright bands, dry holes, hooks, protruberances,* etc. The brilliance of the echoes varies with the intensity and proximity of the phenomenon.

A mathematical equation, expressing the interception of raindrop by a radar beam is given by

$$P_r = C \cdot \frac{Z}{r^2} \qquad ...(7.2)$$

where P_r = average echo power

Z = radar echo factor given by Eq. (7.3)

C = a constant.

The *radar echo factor* (Z) is related to intensity of rainfall in mm/h by the Eqn. (7.3), as :

$$Z = a \cdot I^b \qquad ...(7.3)$$

where a and b are constants for the given radar station, and are to be determined by calibration with the help of recording rain gauges. Typical values of a and b are 200 and 1.6 respectively, for some particular weather radar. The factor Z for this radar is given by :

$$Z = 200 \cdot I^{1.6} \qquad ...(7.3\ a)$$

Proper *attenuation* of the transmitted signal governs the selected radar frequency. Say for example, smaller wave length radars are required to provide better attenuation due to clouds and lighter rains, as compared to those required for attenuating heavy flood producing rains. Hence, radars of 3 or 5 cm wave lengths are generally used for detection of clouds, lighter rainfalls, or snowfalls ; whereas, 10 cm radars may suffice for detecting heavy rains and cyclone surveillance.

For studying the *micro structure of clouds* and detection of *clear air turbulences (CAT),* **laser probes** *(LIDAR)* are increasingly being employed these days. *On line processing of radar data on a computer, and applications of* **droppler type radars** for measuring the velocity and distribution of raindrops, are other advancements, being used in modern days in this field.

Limitations of weather radars. Inspite of their varsatality, weather radars suffer from certain inherent **limitations,** as pointed out below :

(*i*) Distant thunderstroms and precipitation areas may not get detected due to curvature of the earth. The hydrologic range of the radar is about 200 km.

(*ii*) A huge thunderstorm area near the radar station can obliterate the cells beyond it, due to attenuation. The shape of the echoes may get distorted, and may not represent the true extent, shape, and area of the phenomenon.

(*iii*) The intensity of the echo may not correspond to the severity of the weather features. Sometimes, spurious echoes, called **angels** may form due to extraneous factors, such as *hydrolapse, flocks of birds,* etc., and may be misleading. Similarly, echoes always appear from the ground features and topography, and they need to be suppressed by adjustments and intelligence, to highlight the real echoes caused by a particular weather phenomena. *Iso-echo contouring facilities* and other devices installed in the radars do help in suppressing extraneous echoes, and in identifying the particular weather phenomena with greater precision and clarity.

7.8.2. Weather Satellites. The first ever weather satellite of the world was orbitted by USA in the year 1963, and since then, several other improved versions, and *multi-purpose satellites,* have been launched, and put into orbits, by several countries. These satellites continuously provide the meteorological observations, through the process of **remote-sensing** consisting of *imaging* and *communication.* Several countries, including India, are now-a-days, having their exclusive satellites, for providing exclusive data.

A satellite, as you know, is a device or a vehicle, which is launched into the space through powerful rockets, with the injection velocity not less than 11 km/sec, so as to enable it to escape the Earth's gravity and become a **space probe.** The satellite after its

launch, is placed in the *chosen orbit,* called the *parking orbit,* by manoeuvres from the ground control, in order to suit the particular requirements of the country, launching the satellite. Say for example, a satellite placed in a *polar orbit* (*i.e.* an orbit at right angles to the Earth's equator and passing through the polar axis), can scan the maximum area of the Earth's surface. Such *polar satellites* can cover the globe surface twice a day, and hence, such a satellite will send pictures of the same strip of Earth at fixed times, twice a day.

Geostationary or geosynchronous satellites are those, whose orbitting period is 24 hours. Such a satellite will, therefore, revolve round the Earth to complete its one revolution in 24 hours, and hence such a satellite will always remain overhead, *i.e.* in the same relative position with reference to a particular point on the globe, since Earth also takes 24 hours to rotate. The angular speeds of Earth and that of the satellite will be the same, in such a case. This is achieved by putting the satellite in a *geosynchronous circular orbit,* over the Earth's equator, at an altitude of 35786 km. When once the satellite has been put in the *right orbit,* it is placed in the *right slot* at the *decided longitude in the geosynchronous orbit.* Such **parking slots,** are limited, and an international body allots the slot, and your satellite will have to remain within 0.1 degree of the same. This is necessary to avoid interference and over-lapping between satellites. Placing your satellite in a given slot is known as *station acquisition ;* and maintaining it in that position, is known as *station keeping.*

The advantage offered by a geosynchronous satellite is that : the same area of the Earth's surface remains under the constant observation of the satellite all the time ; and this enables the evolution of cyclones, etc., to be studied without interruption.

Theoretically three such satellites, placed at appropriate longitudes over the equator, can watch the entire globe ; but in practice, polar regions will not get covered by such placements, and there will be some overlapping, causing distortions in pictures. Five such satellites, can however, provide practically full coverage of the entire Earth's surface. Indian satellites, INSATs, belong to this category of satellites, as they are *multipurpose geosynchronous satellites.*

A satellite provides us an *overview of the Earths' surface and that of the atmosphere* covering a vast geographical area at any given time, through taking and sending us the integrated pictures or photographs. Such continuous pictures obtained with the passing time, need to be carefully processed and analysed in comparison with the data obtained from other sources. Say for example, *cloud pictures,* transmitted by the satellite as *cloud imageries,* must be correlated with those obtained from other sources, such as *aircrafts.* Similarly, *cyclone formations,* as seen by the satellite from above, may have to be correlated with the ground realities, by *aircraft reconnaissance flights,* etc., to pinpoint the exact cyclone centre. The satellite data, thus, needs to be validated with the aid of ground truth, before it is put to actual use. The satellite observations, thus cannot replace the conventional observations, but can only be complementary to them.

Inspite of their data being only complimentary in character, such satellites provide a very versatile and powerful tool to the meteorologists, who can now have an instataneous panormic view of the entire globe on a continuous basis. Cyclonic storms can, thus, be detected and tracked right from their genesis and fronts, without fail. *Jet streams*, Easterly waves,* etc., are also pinpointed with great success. Satellites can also interrogate unmanned data collecting platforms, constant level baloons, etc., and transmit their data to the main *Data Acquisition Centre,* for analysis and processing.

* *Jet streams* are the high velocity winds, at high altitudes of 16000 m or so, flowing from West to East; helping fast moving air flights, such as from Gulf countries to Japan and Pacific. Reverse flights, against the winds, to be avoided at such heights, where Jet streams exist.

The **body of a satellite** can be broadly divided into three components; *viz* :

(*i*) *pay load systems* (vol ≈ 15%)

(*ii*) *subsystems of the satellite* (vol ≈ 25%) ; and

(*iii*) *fuel systems* (vol ≈ 60%).

(*i*) and (*ii*) together are referred to as *dry mass* of the satellite.

Pay load system is that part of the satellite, which performs its *major service functions,* such as *communication, imaging* and *data relay.* **The subsystems** provide smooth and efficient functioning of the pay load systems by creating the right environment and giving necessary engineering support.

The pay load systems, as pointed out above, provides *communication and imaging.* Functionally, the *communication system* receives signal from the ground at one frequency, amplifies it, and transmits it on another frequency. The tranmission and reception is done at *super high frequencies* (*i.e. C* band, 4000—6000 megahertz) to increase the signal carrying capacity through special antennas. These antennas are made up of ultra high stiffness carbon fibres, and have to be oriented continuously towards the earth station on ground.

The *imaging system* consists of : (*i*) *Very High Resolution Radiometer* (VHRR), which generates the images ; and (*ii*) the *data relay transponders.* The *radio-meter* maps the Earth's surface in the visible and infra-red (I.R.) bands during the day ; and in the infra-red band alone at night, by means of highly sensitive detector elements. The *detectors* need to be kept 'cooled' at about minus 150°C, and, this is achieved by a passive cooler, which isolates the detector assembly, and radiates the heat of the cold space outside.

The reflected/emitted energy from the Earth's surface, is focussed through suitable optics on detectors, which convert it into electrical signals. These signals are then amplified, processed, and transmitted to ground as *digitized signals, making up a single frame, every 33 minutes.* As a detector forms a single picture element, a mirror with *two axis scanning mechanism,* is used to gather the energy from different portions of the Earth, to form a frame. The scanner can be commanded from the ground, to image either the full Earth disc (20°) or a small sector of it (0.5°).

The *cloud pictures* taken by the satellite, together with the information they provide about *winds in the upper atmosphere,* the *sea-surface temperature, precipitation* and *on-going long wave radiation,* are used by the concerned Meteorological Department for *routine weather forecasts,* as well as for developing *weather prediction computer models.*

The *fuel system* of the satellite is so designed as to tranfer and raise the satellite, through its propellant system, into the desired circular orbit of 35786 km over the equator, after initially putting it into some egg-shaped *transfer orbit* (say 200 km × 35786 km) through rocket launching. Say for example, India's satellite—INSAT–2A carried a *bipropellant system,* in the form of an *Apogee Boost Motor*(ABM), and four powerful *thrusters* for emergency. About 400 kg of mono-methyl hydrazine fuel, and 650 kg of nitrogen tetroxide (N_2O_4) oxidiser, was carried on board, in specially made titanium alloy *fuel tanks. High pressue helium gas,* which expels the fuel under zero gravity conditions, was also stored. When once the satellite was launched in a transfer orbit of 200 km × 35786 km, it was raised by *three firings* from ABM, for a total duration of 70 minutes, to kick the satellite from 200 km to 35786 km. *Small thrusters* on board were then activated to make the orbit exactly cicular, and to firmly anchor it in the assigned slot of 74° East. The four emergency thrusters were also provided to raise the satellite, in case the ABM failed. The use of these thrusters would, however, have depleted the fuel and reduce the **satellite life** from nine years to five years.

This INSAT satellite is to draw power from the sun, using *solar cells** in the form of 5 solar panels, having a total area of 15 sq m. A chargeable nickel-cadmium battery system is also provided to store solar energy, to meet its requirement during the time, the sun becomes unavailable to the satellite.

7.8.2.1. India's INSATs. India launched its first ever service satellite, called INSAT–1A in August, 1983, which developed a series of snags, and hence could not be put to use. *INSAT–1B* launched a little later, however became fully operational, and served India's needs quite well for about 10 years. INSAT–1C again failed, because of the difficulty in opening its solar panel. *INSAT–1D* was launched quite late in June 1990, and became fully operational in the parking slot of 83° East. With the launching of INSAT–1D, which worked well till about the year 1998, the first phase of INSAT programme was completed.

The second phase of this programme was started with the successful launching of *INSAT–2A*** on 10.07.1992. *INSAT–2B* was then successfully launched on 23.07.1993, and placed in the old parking slot of INSAT–1B which was shifted and shut down. **INSAT–2C** was then successfully launched on 08.12.1995, and is co-located with INSAT–2B in the same parking slot at 93.5 degrees East.

INSAT-2D, which was subsequently launched in June 1997, however, had to be soon abandoned due to excess loss of power in its placement and manoeuvring. The compensation amount received from the Insurance Company on the failure of INSAT-2D, however, enabled the ISRO to purchase a replacement satellite *Arabsat-IC* (with 26 transponders), which was renamed as **INSAT-2DT**. This satellite was placed in parking slot of 55° East, and continued to work till Oct. 2003, when it was replaced by INSAT-3E.

INSAT-2E, after several design improvements (to take care of the causes leading to the failure of INSAT-2D), was then successfully launched on 03.04.1999, and was co-located with INSAT-ID, which was shut off completely. This satellite is placed at 83° East along with INSAT-1D & INSAT-2D, which are totally shut off. INSAT-2E is the last satellite of INSAT-2 series.

The first of the third generation INSAT-3*** series satellites, *i.e.* the **INSAT-3B** with 12 UXC band transponders, was successfully launched by Ariane-5 launch- vehical on 22.03.2000, well before the launching of INSAT-3A, due to severe demand and shortage of UXC band (upper extended C band) transponders required in television services. INSAT-3B has also been co-located with INSAT-2E at 83° East, and is working well today.

The second satellite of INSAT-3 series, *i.e.* **INSAT-3C**, weighing 2750 kg, was launched on 24.01-2002, successfully. This INSAT, with a designed mission life of 12 years, has replaced the aging INSAT-2C, to give impetus to India's telecommunication, broadcasting, business communication and mobile services.

The third setallite of this series **INSAT-3A** weighing 2950 kg, was, then launched on 10.4.2003, successfully by Arian-5 launch-vehicle from Korou in Franch Guyana, to replace old INSAT-2A, to boost country's communication and meteorological services.

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* Basic unit is a small rectangular *silicon chip* (2 cm × 4 cm) converting about 12-13% of the incident solar energy into electrical energy by using **photo voltaic effect.** The silicon solar cells used in most of our Earth satellites have, however, been replaced in INSAT-2E, by high efficiency gallium arsenide/germanium based solar cells, which have a solar power conversion efficiency of 18% as against the 12-13% of silicon cells.

** Truly speaking, INSAT-2A, was only notionally operational with none of its transponders being capable of providing any service due to depletion of on-board propellent, caused by some problem with its propellant tank design from April, 1998 onwards. It has now been replaced by INSAT-3A.

*** The figs-1, 2, 3..... etc. in INSATs usually refer to the satellite mass in tonnes. Thus, the Fig. 1 in INSAT-1' series refer to 1 tonne class of satellites ; and the Fig 2 in INSAT-2's refers to the 2-tonne class of satellites. The INSAT-2A thus had a mass of 2 tonnes, while INSAT-2E had a mass of 2.55 tonnes. INSAT-3 class, launched recently are heavier satellites, each having a mass of the order of 3 tonnes. The latest launched INSAT-3E, Infact, had a mass (lift of mass) of 2.75 tonnes. *Note : Heavier satellites need heavier cryogenic engines for launching.*

The fourth satellite of INSAT-3 series, i.e. INSAT-3E, weighing 2,750 kg was launched on 28-09. 2003, with a designed mission life of 12 years, and to replace INSAT-2DT, to give a boost to telecommunication and television services. This INSAT was also launched by an European Ariane-5 rocket from the spaceport of Kourou in French Guyana and injected into the geosynchronous transfer orbit in 29 minutes of its flight in multistep intricate mission.

Most of our INSATs are multipurpose geosynchronous satellites rendering *three types* of services ; *viz :*

(1) **Meteorological services,** including the supply of weather maps, cyclone and tornado movements, cloud imageries, etc.; including *Relay* of meteorological, hydrological, and oceanographic data, (collected by *unmanned Data Collection Platforms* (DCPs) located in remote uninhabited places) to the *Meteorological Data Utilisation Centre* (MDU Centre) at New Delhi, for further processing.

INSATs are also being used to *activate Disaster Warning Systems* (DWS), installed in several cyclone-prone villages of Andhra Pradesh and Tamil Nadu coasts, regarding the approaching cyclones. Such a DWS system, on being activated by the *Cyclone Warning Centre of Madras, via,* INSAT, emits a siren for one minute, giving ample warning to the area residents, regarding the approaching cyclone.

As a matter of fact, such centres are purely the reception centres, receiving coded messages from the cyclone warning centre, through INSAT ; and are even provided with public address system, so that an announcer from the cyclone warning centre can inform the people, about the time as to when the storm is expected to hit them, and what action to take. Such a DWS, hardly costs around Rs. 1 lakh, but their installations have tremendeously helped in restricting loss of life during cyclones. We have already installed more than 100 such systems in Andhra and Tamil Nadu areas ; and we plan to further install such systems in West Bengal, Orissa, Gujarat, and Maharashtra's coastal areas. Even Bangladesh, where cyclones cause heavy human loss, may install such systems, as we have already offered this service to that country.

(2) **Telecommunication services**, including improved voice and quickly fructifying STD and international telephone calls ; and commissioning of many such new routes. INSAT system has also provided a novel rural tele-graphic service in north-eastern inaccessible areas of the country, where the use of conventional wire telegraphy becomes impracticable, due to hilly terrain.

(3) **Television services,** including much wider coverage of transmitted T.V. programme throughout the country, as well as much wider reception from around the world.

INSAT–2A, besides giving the above services, had six additional transponders in expanded C–band to provide **business communication, satellite news gathering,** and **distance education,** using *talk-back system.*

Weather pictures were expected to be clearer from INSAT–2A as compared to those obtained from INSAT-1B, because of the enhanced resolution of the on-board radiometer*. One additional feature of INSAT–

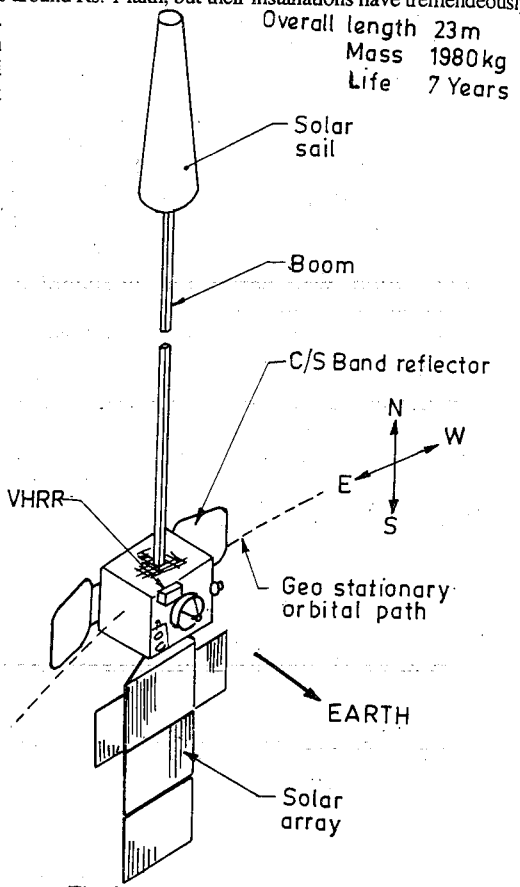
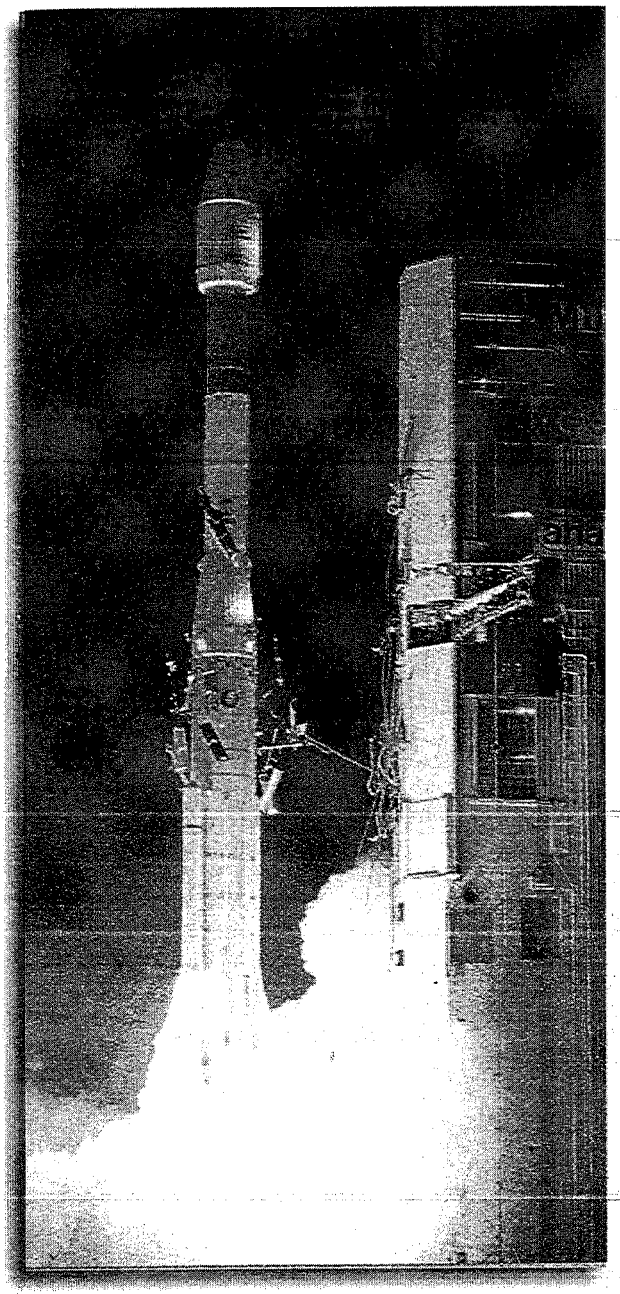
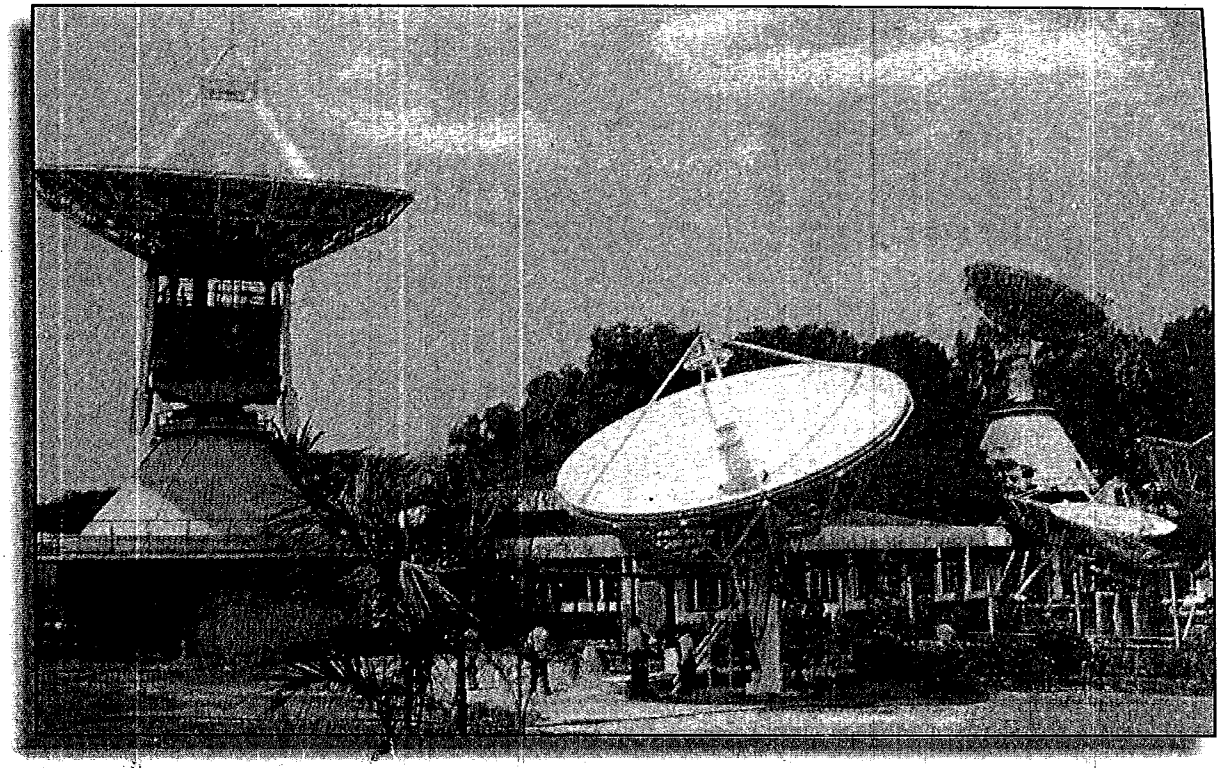


Fig. 7.7 View of INSAT-2A (in orbit).

* The resolution was improved in INSAT–2A to 2 km in the visible band (from the earlier one of 2.75 km in INSAT–1B) and to 8 km in infra-red band (from the earlier one of 11 km).



The European launcher Ariane 42P blasts off from Kourou in French Guiana on April 3, 1999 to deploy INSAT-2E. The multi-purpose satellite is perhaps the most advanced in the INSAT series so far in terms of technologies used in the hardware and in the payloads.



An array of full-motion antennae and limited-motion antennae at the Master Control Facility in Hassan (Karnataka), which controls, commands and tracks INSAT-2E.

2A was its *search and rescue payload,* which could detect distress calls, eminating from emergency radio beacons, carried by ships, aircrafts, or group of mountaineers, etc. at 406 megahertz frequency. At present, such *distress call detection work* is being done by polar satellites of USA, orbitting in *low-Earth orbit,* around the world. Such a satellite, after detecting an emergency distress call, passes it on to a nearby *Local User Terminal Centre* (LUT)*, for helping in providing the necessary assistance to the distressed.

The detection and rescue operation carried out by USA some years back, in saving the lives of PLO chief–Mr. Arafat and others, whose hellicopter crashed in a desert, is an excellent testimony of the use of satellites in detecting such emergent needs.

Although *such distress-call detection work* is being done at present, by low earth polar satellites ; but near the equator, there could still be a delay of $1\frac{1}{2}$ to 3 hours, between the time an emergency radio beacon is activated by the distressed, and the time at which it is detected. Such a possible delay in detecting emergent signals, may prove disastrous to the distressed. This time gap can be eliminated by geostationary satellites, which will be in a position to detect such distress calls immediately, in their respective areas, being watched. The use of such rescue pay loads on geostationary satellites, is still experimented globally and the first Indian satellite equipped with this facility was INSAT-2A, which unfortunately has stopped working from April 1998.

A prospective view of INSAT–2A is shown in Fig. 7.7. Similarly, Fig. 7.8 shows a line eye view of the services, performed by it.

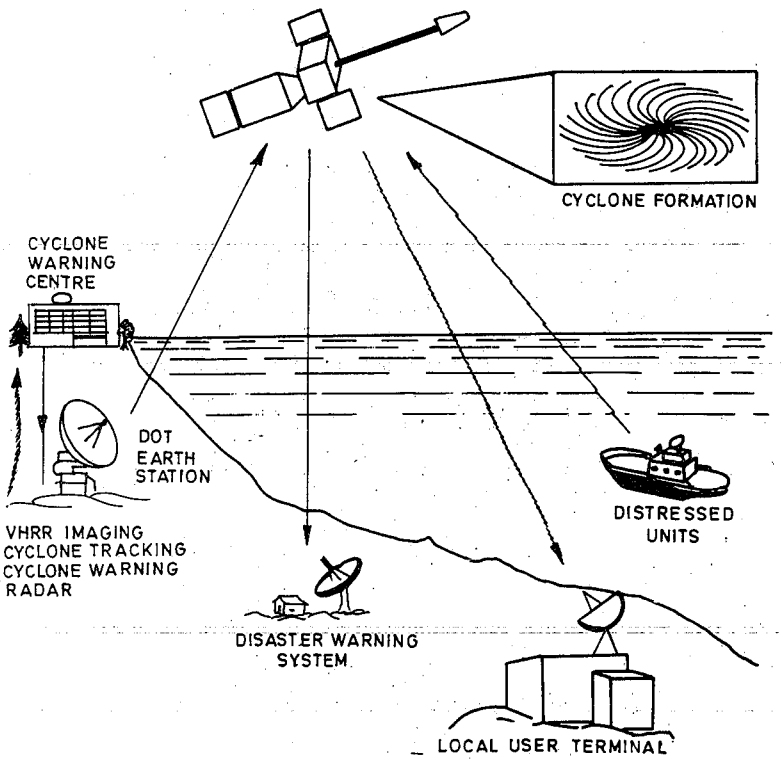


Fig. 7.8. Services performed by INSAT-2A.

INSAT programme in India is being conducted and controlled by *Indian Space Research Organisation* (ISRO). All the satellites in INSAT-1 series were, however, built

* In India, we have two LUTs ; one at Bangalore and one at Lucknow.

by *Ford Aerospace* of USA, as per design specifications provided by ISRO. These satellites, after being brought from USA, were launched and parked in their parking slots, by American scientists stationed in Hassan near Bangalore. INSAT-2 series has however been indigenised to a large extent. Say for example, INSAT-2A was indigenised by about 40% and when once it was launched into the elliptical transfer orbit (200 km × 35786 km) by the Ariane rocket of *European Space Agency* from French Guyana, the raising and parking job was done by the Indian scientists and engineers of ISRO from the *Master Facilities Centre*, Hassan (Karnataka), from where commands are sent to INSATs. INSAT-2B, INSAT-2C, INSAT-2D as well as INSAT-2E were further indigenised and improved to give better performance on all fronts. *INSAT-2C and INSAT-2D (which had failed), however, were not provided with any meteorological transponders to relay weather data, and were aimed only at the enlarging and extending the television and telecom services even beyond Indian boundaries.* True to this aim, ISRO has leased nine transponders to INTELSAT (an international and inter-governmental space consortium created in association with 133 countries) under an agreement signed on 30.01.1995. This lease agreement fatches us $ 10 million (≈ Rs.42 crores), annually.

INSAT-2E has, however, been provided with sufficient improved meteorological pay loads, and is likely to enhance the meteorological services, which are being provided at present only by INSAT-1D, since these functions are now no longer being performed by INSAT-2A (which has already become inoperational due to depletion of on-board propellent), and INSAT-2B (due to burning of the electrical wiring of these parts), and since INSAT-2C and INSAT-2DT (replaced by INSAT-3E) do not have any meteorological pay loads.

From meteorological point of view, therefore, INSAT-2E (launched in June 1999) becomes the most important INSAT, as it has become the only INSAT providing such facilities after it has replaced INSAT-1D, which has been shut off, as stated earlier.

Considering the importance of INSAT-2E for continuing and enhancing the meteorological services, extra care has been taken for its success by removing the shortcomings of INSAT-2D and launching it in a larger elliptical orbit of 250 km × 36,155 km, thereby increasing perigee to 250 km from the earlier figure of 200 km to save on the on-board fuel, in raising it to the circular orbit of 35786 km.

INSAT-2E also carries improved meteorological pay loads, as it carries an improved version of the *Very High Resolution Radiometer* (VHRR) and a high resolution *Charge Coupled Device (CCD) Camera.* Besides the usual visible and thermal infra-red (I.R.) band, the VHRR will have a new *water vapour channel.* In addition to imaging cloud cover including the formation and movement of cyclones (as was being done earliear by INSAT-1D), the water vapour channel will aid in estimation of water vapour in the clouds, which will help in forecasting of rains in a better way. This will greatly help in precise and more accurate study of monsoons. Also, for the first time in India, this will enable the estimation of upper tropospheric winds, blowing at 8-12 km above the ground level. The CCD camera is an off shoot of the successful experience of working with such cameras in the Indian Remote Sensing (IRS) satellites. This camera operates in three bands — visible, near infra-red, and short wave infra-red ; and with its higher resolution of 1 km as against 2 km of VHRR, its data will compliment the VHRR data and assist in analysing cyclones, monitoring local severe storms, and in making better and reliable forecasts of heavy rainfall, etc.

In compliments to such expanded facilities in weather detection and warning made feasible by INSATs. IMD has developed a *main data utilisation centre* (at New Delhi); 20 subsidiary centres ; a network of nearly 100 unattended data collection platforms (DCP) ; and several *disaster warning systems* (DWS), spread around the country to suit local needs. The system has proved to be a great boon to forewarn and thus to forearm the public, especially settled in coastal areas of Andhra Pradesh and Tamil Nadu, to mitigate their miseries from perpetual cyclones. Several such DWS are being planned to be installed in

coastal areas of Orissa, West Bental, Gujarat and Maharashtra, as pointed out earlier. Just as INSAT-2E is mainly being used for meteorological services, INSAT-3B on the other hand, is primarily to be used for business development & mobile communications. This has helped in improving VSAT* services, being used by about 300 corporate companies, Banks, stock markets, white goods sector, medium to heavy engineering companies, among others including telemedicinal & distant educational uses. INSAT-3E, on the other hand, is primarily meant for serving telecommunication and television services in the country.

The expenditure figures on INSATs are also quite high. The building cost of INSAT-2A and INSAT-2B was around Rs. 243 crores each, with an additional cost of around Rs. 161 crores for launching. Similarly, the total cost of INSAT-2E was about Rs. 550 crores, which included Rs. 286 crores paid for the launching to the Arianespace (French company) and the cost paid for insurance, etc. The cost of INSAT-3B is also approximated to be Rs. 500 crores including launching cost of Rs. 276 crores & insurance cost of Rs. 66 crore. *Be that as it may, no cost is high to modernize our systems, and to acquire such modern technologies, as it ultimately leads to enormous savings, in the form of prevention and protection of invaluable life as well as properties.*

PRECIPITATION GAUGES AND PRECIPITATION DATA

7.9. Types of precipitation

Although the moisture is always present in the atmosphere, but it is condensed only when the air is cooled, so that it becomes saturated with the same amount of water vapour. The usual mechanism by which the air is cooled to cause precipitation is the lifting of the air mass. There are three different methods by which the air mass gets lifted, so as to cause cooling and condensation of the atmospheric water vapour, and the subsequent precipitation mainly in the form of rain or sometimes under special conditions as snow, hail, sleet, etc. Depending upon the way in which the air is cooled, as to cause precipitation, we can have three kinds of precipitation, as given below :

7.9.1. Cyclonic Precipitation. It is caused by the lifting of an air mass due to the pressure difference. If low pressure occurs in an area, air will flow horizontally from the surrounding area, causing the air in the low pressure area to lift. The precipitation that results, is called **non-frontal cyclonic precipitation.** If one air mass lifts over another air mass, the precipitation is called **frontal cyclonic precipitation.** The boundary between these two air masses of different temperatures and densities (one warm air mass and the other colder) is known as a **front** *or frontal surface.* The large whirling mass of air, at the centre of which the barometric pressure is low, is known as a **cyclone.** The air that rushes horizontally into the low pressure area changes into a whirling mass because of the rotary motion of the Earth about its own axis. The cyclone is a very large mass of air ranging from 800 to 1600 km in diameter and moving with a velocity of about 50 km/hr. The central portion of this cyclone, where the pressure is low, acts like a chimney, through which the air gets lifted, expands, cools and finally gets condensed causing precipitation.

Cyclonic precipitation can occur in the form of drizzle, intermittent rain, or steady rain. If the precipitation is caused by a *cold front,* it is very intense and of short duration; while that caused by a *warm front* is more continuous. Third possibility is that of an occluded front. An *occluded front* occurs when a cold front over-takes a warm front. The precipitation pattern is a combination of both cold and warm frontal distributions. A cold front is that in which the warm air replaces the colder air, whereas in a warm front the case is opposite.

Cyclones are responsible for most of the rains in the central part of the United States, and for most of the winter rains in Haryana and Punjab in India.

7.9.2. Convective Precipitation. It is caused due to the upward movement of the air that is warmer than its surroundings. Generally this kind of precipitation occurs in

* Very small Aperture Terminal.

tropics, where on a hot day, the ground surface gets heated unequally, causing the warmer air to lift up ; as the colder air comes to take its place. The vertical air currents develop tremendous velocities and are hazardous to aircrafts. Precipitation occurs in the form of showers of high intensity and short duration.

7.9.3. Orographic Precipitation. It is the most important precipitation and is responsible for most of the heavy rains in India. Orographic precipitation is caused by air masses which strike some natural topographic barriers like mountains, and cannot move forward and hence rise up, causing condensation and precipitation. The greatest amount of precipitation falls on the windward side, and the leeward side often has very little precipitation. Orographic barriers tend to increase both cyclonic and orographic precipitation because of the increased lifting involved. The rainfall is composed of showers and steady rainfall. A striking example of this kind of natural barriers is the southern slope of Himalayas in India. The winds, heavily laiden with moisture from the Bay of Bengal, strike the southern slope of Himalayas, causing intense rains ; so much so that in Cherrapunji, the average annual rainfall is of the order of 1270 cm. Similarly, the winds coming from the Pacific Ocean strike the western slopes of the Coastal ranges in Washington U.S.A., causing heavy rains there.

7.10. Measurement of Rainfall by Rain Gauges

In order to estimate the effects of precipitation, it is necessary to measure the precipitation, and to find out its distribution at various places on the earth.

All forms of precipitation are measured as the vertical depths of water that would accumulate on a level surface, if the entire precipitation remained where it fell. The total amount of precipitation falling on Earth in a given period, will hence be expressed as the depth to which it would accumulate on the *horizontal projection* of the Earth's surface, if there were no losses by evaporation or runoff and if any part of precipitation falling as snow or ice were also melted. The two important parts of precipitation (*i.e.* rain and snow) are measured separately by measuring devices, called rain gauges and snow gauges.

Since the amount of precipitation varies from place to place, it is necessary to install measuring devices at various key points. The simplest method of measuring precipitation is by setting up gauges with a horizontal circular aperture of known area, and collecting and measuring at regular intervals the precipitation collected in them. It is assumed that the amount of precipitation collected in the gauge is representative of a certain area around the point where the measurement is made.

7.10.1. Types of Rain-gauges. Any open receptacle with vertical sides can be used as a gauge for measuring rainfall. These refined receptacles, which are used for measuring rainfall, are called *rain gauges*. Two kinds of rain gauges, are generally used. First is the 'non-recording type' rain gauges, and the second is the 'recording type' rain gauges. These two types of gauges are described below :

(1) **Non-recording rain gauges.** Non-recording rain gauges are most widely used in India. They are known as 'non-recording type' because they do not record the rain, but only collect the rain. The collected rain is then measured by means of graduated cylinders, so as to directly represent the rainfall volume in cm of water depth.

i.e.
$$\left[\frac{\text{Volume of water collected in cm}^3}{\text{Area of the aperture of the gauge in cm}^2} = \text{cm depth of rain fallen}\right]$$

Various models have been designed for such gauges ; out of them, the **'Symon's type'** (Refer Fig. 7.9) was most widely used in India till 1969. However, since then, the

Indian Meteorological Department has adopted another model called **Standard gauges** for their use at all rainfall stations in this country. The Symon's and other such types of rain gauges have, therefore, been replaced in India, and have become obsolete.

The complete installation of the new Standard type of non-recording rain gauge, which is now being used in India, is shown in Fig. 7.10. Such a rain gauge consists of a collector, with a gun metal rim, a base, and a polythene

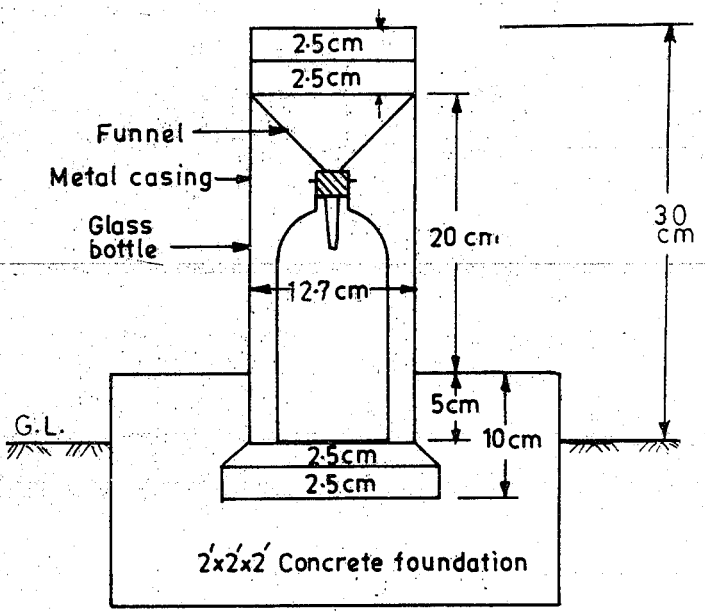


Fig. 7.9. Symon's non-recording rain gauge.

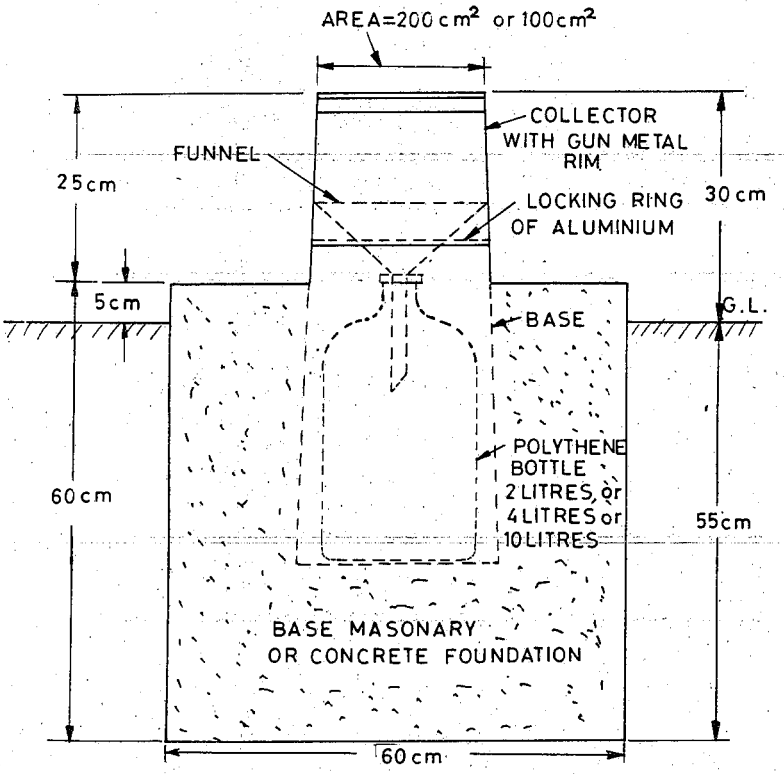


Fig. 7.10. Standard non-recording Rain-gauge of Indian Meteorological Department.

bottle. Both the collector and the base are made of fibre glass reinforced polyester. The collector has a deep set funnel, and the complete rain gauge has a slight taper with the narrower portion at the top, as shown. The collectors have either apertures of 100 or 200 sq cm area (*i.e.* 112.9 mm or 159.6 mm dia) and are so made that they are interchangeable. There are also two types of interchangeable bases, the smaller base being used for all sizes of rain gauges except the largest. The polythene bottles are of three sizes having capacities of 2, 4 and 10 litres of water. The possible combinations of three essential components provide the following capacities of rain-gauges :

Table 7.4

S. No.	*Collector area in sq cm*	*Base size*	*Bottle capacity in litres*	*Normal capacity of rain gauge in cm of rainfall*
1	200	small	2	10
2	200	small	4	20
3	100	small	2	20
4	100	small	4	40
5	200	big	10	50
6	100	big	10	100

Of the above six possible combinations, those at serial numbers 1, 2, 4 and 6 are considered to be sufficient to meet the rainfall conditions in our country. Thus, the new standard rain gauges may have capacities of 10, 20, 40 and 100 cm of rainfall. It is worthwhile to note that the capacities of the rain gauges can be altered by changing either the collectors or the polythene bottles inside them. The 20 cm capacity rain gauge, with 200 sq cm collector and 4 litre bottle, is most widely used, and is sufficient to measue the 24 hours rainfall of most of the Indian stations. The 40 and 100 cm capacity gauges, being used only at few places, having heavy to very heavy rainfall, and 10 cm capacity gauges being used at places having very scanty rainfall.

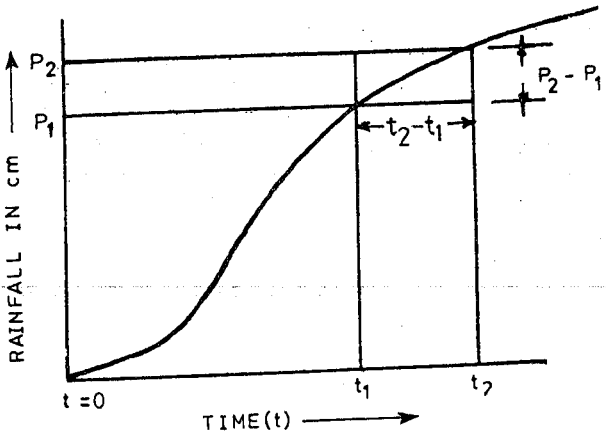
All these standard rain gauges, irrespective of their capacities, can be installed at a standard height of 30 cm above the ground level, which was not possible with the old Symon's types.

The collector and the base are locked to each other by means of two complementary locking rings, fixed inside the collector at its lower end and the base at its top end.

When the rain falls, it is collected in the bottle. A man comes daily at 8.30 A.M. (in India) and measures and records the rainfall collected in the bottle by pouring it into a standard graduated measuring jar (two different types of jars for 200 cm^2 and 100 cm^2 collectors are available). *This process of measuring at 8.30 A.M. and recording the rainfall of the past 24 hours, is common throughout this country.* However, if the rainfall on a certain day is too much, and is likely to exceed the capacity of the bottle, then two or three intermediate readings may be taken, and their sum will then have to be recorded as the rainfall of the past 24 hours of the day, on which the final reading (*i.e.* at 8.30 A.M.) is taken. When the rainfall on a given day exceeds 2.5 mm, then that day is called a **rainy day.**

(2) **Recording type rain gauges.** Recording type rain gauges are those rain gauges, which can give us a permanent automatic rainfall record without any bottle reading. In this type of a gauge, a man has not to go to the gauge to measure or to read the amount of rain fallen. There is a mechanical arrangement by which the total amount of rain fallen, since the record was started, gets recorded automatically on a graph paper. The gauge, thus, produces a record of cumulative rain *vs.* time, in the form of a graph, which is known as the *mass curve* of rain fallen, and is shown in Fig. 7.11

Besides giving the total amount of rain fallen at a station, such a curve will also help in indicating the times of onset and cessation of a rain, and, thereby indicating its duration. Moreover, the slope of the curve gives us the intensity of rainfall for any given period, as indicated in Fig. 7.11. Since such gauges represent the cumulative rain, they are sometimes called as *integrating rain gauges.* Moreover, such gagues can provide us with continuous recorded measurements for a number of days. They are of great utility in hilly and far off areas, where it is not practically feasible to daily visit the gauge station. Such gauges are, therefore, also sometimes known as *continuous rain gauges.*



$$\frac{P_2 - P_1}{t_2 - t_1} = \frac{dP}{dt} = \text{Rate of rainfall or Intensity of rainfall.}$$

Fig. 7.11. Mass curve of rain.

A recording rain gauge is generally installed in conjunction with an ordinary non-recording gauge exposed closeby. The nearby installed ordinary gauge will serve as an aid, and a standard for checking and adjusting the recordings of the recording gauge.

Various models have been designed for recording gauges ; on the basis of which, various gauges, such as "Tipping bucket type", "Weighing type", and "Floating type" etc. are

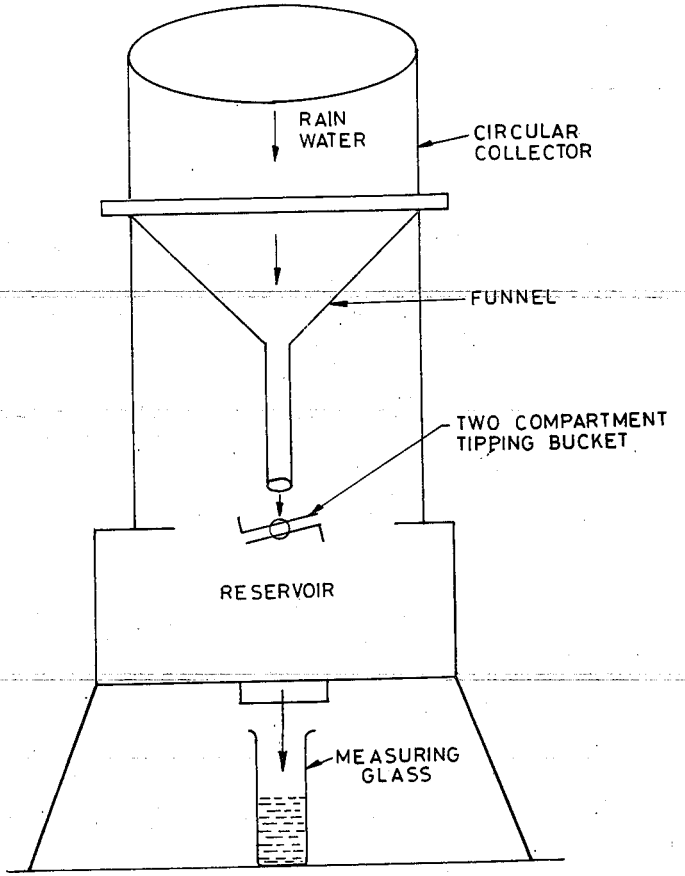


Fig. 7.12. (*a*) Indicating the recording mechanism of a "Tipping Bucket" type of a recording rain gauge.

available in the market. In general, such a gauge consists of a *rotating drum* with a graph paper fixed around it. There is a pen point in contact with the graph paper, which moves up with the collected rain, and thus recording the cumulative rain, with the passage of time.

(*a*) In a *Tipping bucket gauge,* the rain water is first caught in a collector and then passed through a funnel. The funnel discharges the water into a two-compartment bucket. When 0.1. mm of rain water gets filled up in one compartment, the bucket tips, emptying into a reservoir, and moving the second compartment into place beneath the funnel, as shown in Fig. 7.12 (*a*) and (*b*). The tipping of the bucket completes an electric circuit, causing a pen to mark on a revolving drum. Since the movement of the tipping of the bucket can be transmitted electronically over distances, such gauges are generally installed in hilly and inaccessible areas, from where they can supply their measurements directly to the control room at meteorological station. No graph paper or drum etc. in this gauge is installed, and the rainfall measurements are directly recorded at the control room, and the gauges are left without any watch and ward, except periodic repairs and inspections.

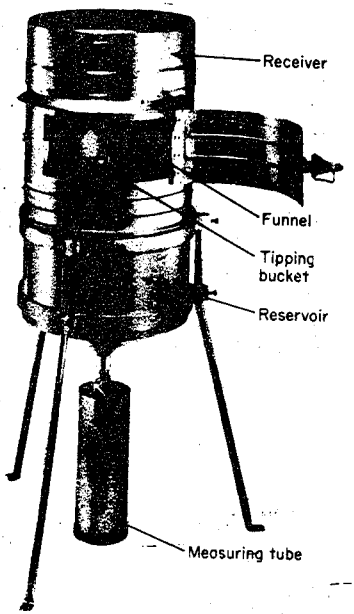


Fig. 7.12. (*b*) Photographic view of a Tipping Bucket type of a rain gauge.

(*b*) The *Weighing type gauge* weighs the rain which falls into a bucket placed on the platform of a spring or a lever balance, or any other weighing mechanism, as shown in Fig. 7.13. The increasing weight of the bucket helps in record-

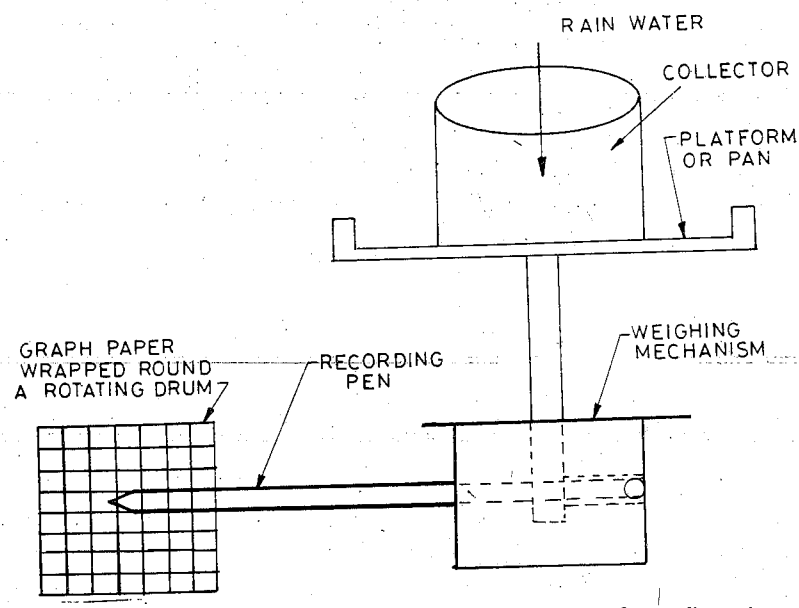


Fig. 7.13. Indicating the recording mechanism of a "Weighing" type of recording rain gauge.

ing the increasing quantity of collected rain, with time, by moving a pen on a revolving drum. Such gauges are generally not used in India, though they are widely used in U.S.A.

(*c*) Various types of *Float recording gauges* are in use these days. In such a gauge, the rise of floating body due to the increasing rain catch, helps in lifting the pen point, which goes on recording the cumulative rain with time on a graph paper wrapped round a rotating drum. Such a gauge may be emptied either manually or by self-starting syphons. A float type gauge, provided with a self-starting syphoning arrangement, is most widely used in India ; and is popularly called *Natural Syphon Recording rain-gauge.* Such a rain gauge is described below in detail :

A natural syphon recording rain gauge is shown in Fig. 7.14 (*a*), (*b*) and (*c*). Different parts are properly named there.

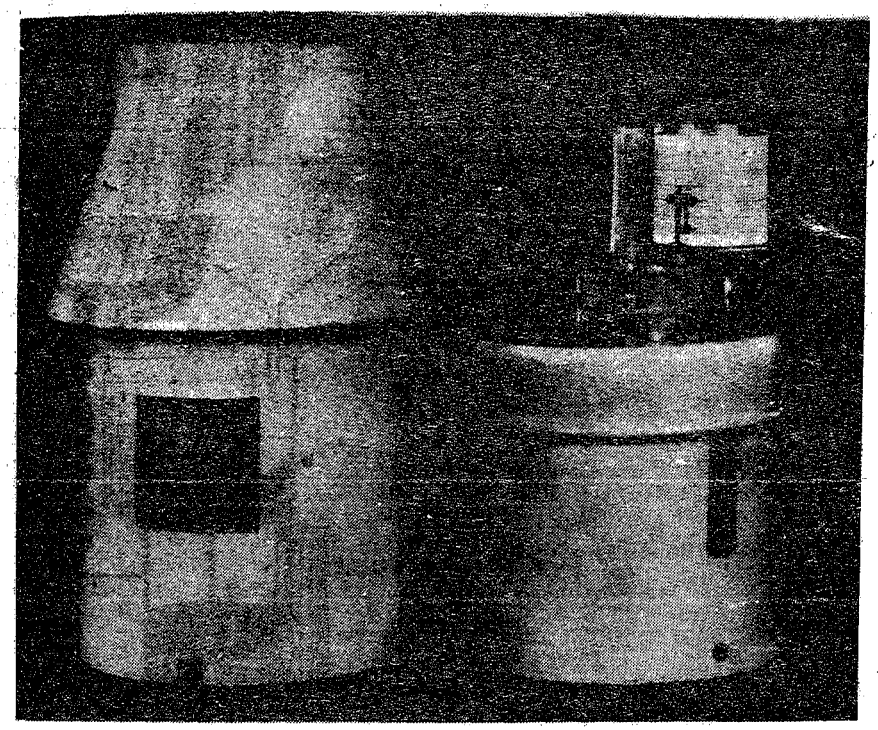


Fig. 7.14. (*a*) Photographic view of an automatic "Float recording" rain gauge popularly used in India.

The rain water entering the gauge at the top of the cover is led via the funnel to the receiver, consisting of a float chamber and a syphon chamber. A pen is mounted on the stem of the float, and as the water level rises in the receiver, the float rises, and the pen records on a chart, wrapped round a clockwise rotating drum, the amount of water in the receiver at any instant. The rotating drum completes one revolution in 24 hours, or sometimes in 7 days, depending upon the gauge, and the graph chart will have to be replaced after this much of time. Syphoning occurs, automatically when the pen reaches the top of the chart, and as the rain continues, the pen rises again from the zero line of the chart. If there is no rain, the pen traces a horizontal line from where it leaves off rising.

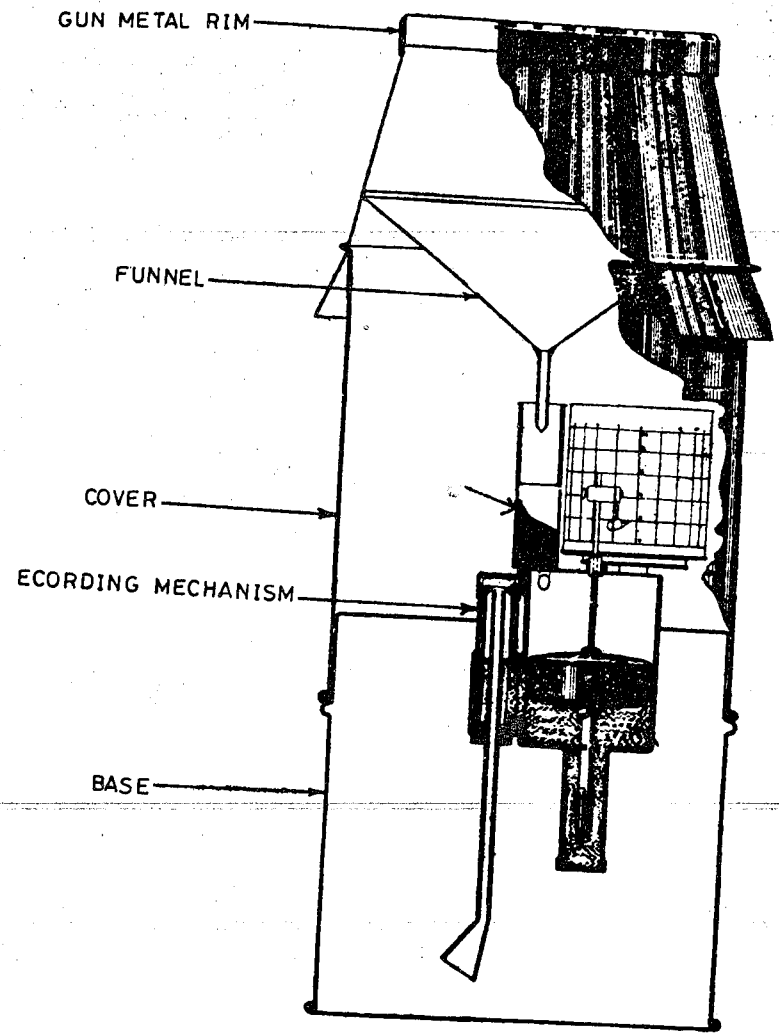


Fig. 7.14. (*b*) Section through an automatic rain gauge (Floating type).

The operation of the natural syphon arrangement is described below :

The syphon is arranged concentrically, the long discharge tube being surrounded by the shorter syphon chamber, which is directly connected to the float chamber. A glass piece is placed over the joint of these tubes. The passage connecting the two tubes at this joint is of almost capillary dimensions, but the sectional area is large enough to discharge the water collected in the receiver, with sufficient speed. The upper end of the water level falls to a certain depth. When syphon ceases to act, the water column is broken at a definite stage by a bubble of air which gets into the capillary, and freedom from dribbling is thus ensured. After syphoning, there is just enough water to float the float.

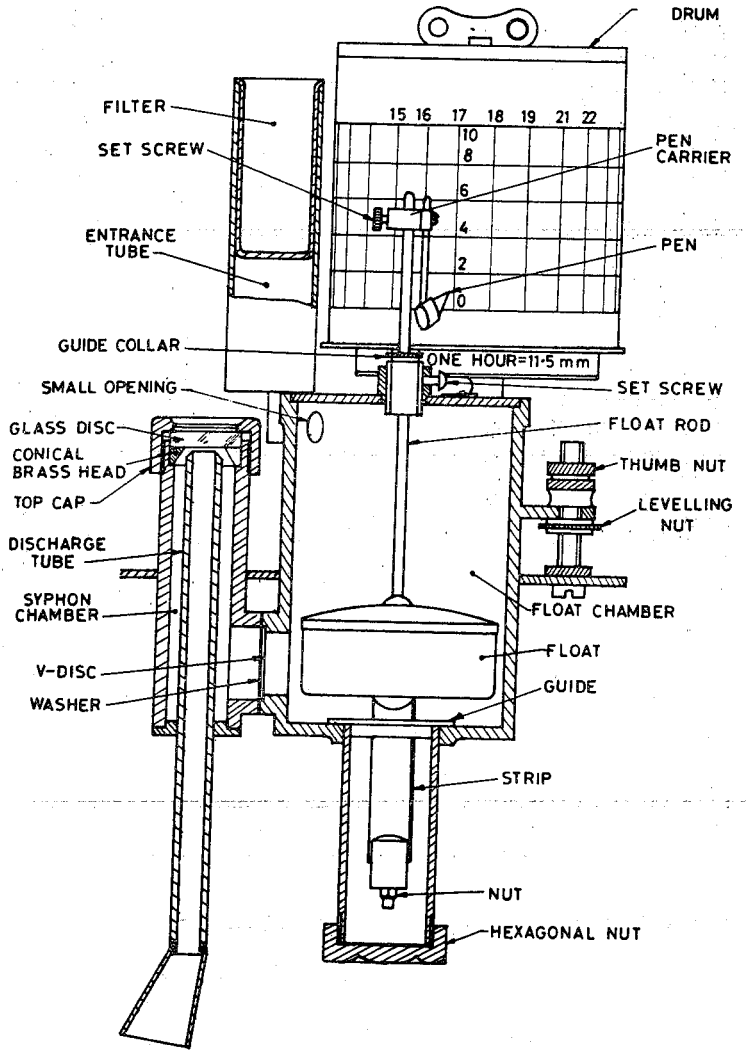


Fig. 7.14. (*c*) Recording mechanism details of Floating type automatic rain gauge.

7.11. Errors in Rain-gauge Measurement and Estimating True Rain-Catch

The amount of rain water collected and measured by a rain gauge may not always represent the exact amount, which would have been caught, if there were no factors trying to err the recorded amount. For example ; there may be instrumental errors in the gauges, or in their recording or measuring arrangements ; some rain water may get lost due to splash from the collector ; some water from an initial rain may get lost in moistening the gauge funnel and other inside surfaces ; blowing winds may tilt the rains from vertical, thus bringing lesser catch in the vertical gauge ; dents in the collector rim may change its receiving area, ; vertical upward air currents may impart upward acceleration to precipitation, thus bringing lesser catch in the gauge ; etc. All such factors try to introduce errors in the measured catches. Some of them may increase the catch,

and some of them may decrease the catch. However, in general, it can be stated that almost all the errors that are introduced in rain catch measurements have a tendency to yield measurements which are too low. In other words, *the observed rain catch needs to be increased for the likely errors introduced in its measurement.*

Of all the possible errors, the most serious error is introduced by wind, which may result in a vertical acceleration of air, forced upward over the gauge. Such upward air current will impart an upward acceleration to the precipitation, about to enter the gauge, thus resulting in a deficient catch. The deficiency is greater for small rain drops than for large drops, and *is thus greater for lighter rains than for heavy rains.* It is not possible to precisely and reliably evaluate the wind error, because of problems involved in determining the true precipitation reaching the ground. Attempts at assessing the wind error, usually consist of comparing gauge measurements with weight changes in nearby lysimeters, or with changes in lake levels, or merely comparing measurements of shielded* and unshielded gauges. The error introduced by wind effects, increases with the wind velocity ; and since wind velocity is more at high attitudes, the gauge installed at higher altitude will definitely catch lesser rain catch. *Higher the gauge, greater will be the wind error, and hence more deficient will be the rain catch.*

Based upon the experimental works, it has now been possible to correlate the wind velocity and the gauge catch deficiency (*i.e.* so many per cent of lesser catch collected) as shown in Fig. 7.15.

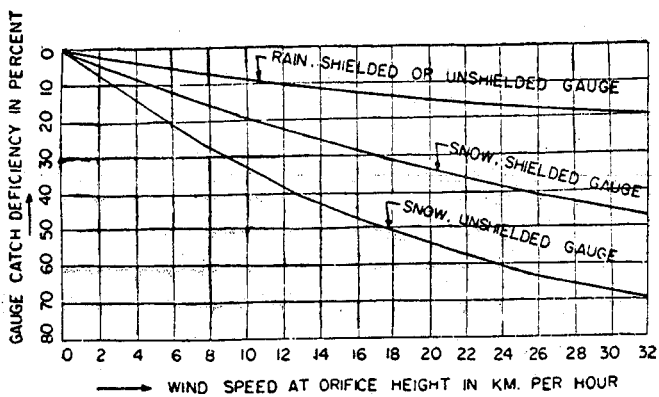


Fig. 7.15

The above chart has been provided by U.S. National Weather Service.

In order to get as correct measurements as possible, which may be free from wind errors, it may be worthwhile to install a gauge on a level ground with bushes or trees serving as *wind breakers,* provided they are not so close as to reduce the gauge catch. Trees or other obstacles serving as wind-breakers should subtend angles of 20° to 30° from the gauge orifice, with none greater than 45°, and should fairly well surround the gauge, so as to provide protection from all directions. An ideal site would be a *coniferous forest.*

Various methods and formulas have been put forward to estimate the true or actual amount of rainfall from the observed rain catch measurements. One of them is based on

* The gauges which are shielded from the effects of winds, etc.

the premise that there is a relationship between the ratio of the unshielded gauge catch (P_u) to the actual precipitation (P), and the ratio of the unshielded-gauge catch (P_u) to the shielded gauge catch (P_s) ; or

$$\log \frac{P_u}{P} = b \cdot \log \frac{P_u}{P_s} \qquad ...(7.4)$$

where, b is a calibration coefficient, depending upon the type of the gauge.

Tilted gauges. When rain is falling vertically, a gauge inclined 10° from the vertical, will catch about 1.5 per cent less than it should. However, if a gauge on a level ground is inclined slightly towards the wind, it will catch more than the true amount.

7.12. Estimating Missing Rainfall Data

Sometimes, the rainfall amount at a certain rain gauge station for a certain day(s), may be missing due to the absence of some observer or instrumental failure. In such cases, it might be needed to estimate the missing rainfall amount by approximating the value from the data of the nearby rain gauge stations. The following methods are generally adopted for computing the missing rainfall data.

Three rain gauge stations as close to and as evenly spaced around the station with the missing record (*i.e.* station X) as possible, are, first of all, chosen. The rainfall data for these three stations (*i.e.* 1, 2 and 3) on the day for which the data at the station X is missing are now collected. The average annual rainfall values at all the four stations should also be known. Now, if the average annual rainfall at each of these three index stations differs within 10% of the average annual rainfall of the station X (*i.e.* the station with missing data), then a simple arithmetic average of the precipitations (corresponding to the missing precipitation) at the three index stations will give us the estimated quantity. Thus, if N_1, N_2, N_3 and N_x represent the average annual rainfalls at stations 1, 2, 3 and X respectively; and P_1, P_2, P_3 and P_x represent their respective precipitation data of the day for which the data is missing at station X; then we have

$$P_x = \frac{P_1 + P_2 + P_3}{3} \text{ [Provided } N_1, N_2 \text{ and } N_3 \text{ differ within 10\% of } N_x\text{]} \qquad ...(7.5)$$

However, when the average annual precipitation at any of the three stations differs from that at the station in question by more than 10%, the normal ratio method is used. In this method, the amounts at the three index stations are weighted by the ratios of their average annual precipitation values. Hence, the missing precipitation data P_x, in such a case, will be given by

$$P_x = \frac{1}{3}\left[P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + P_3 \frac{N_x}{N_3}\right] \text{ [Provided any of } N_1, N_2 \text{ and } N_3 \text{ differs from } N_x \text{ by more than 10\%]} \qquad ...(7.6)$$

Example 7.1. *Precipitation station X was inoperative for part of a month during which a storm occurred. The respective storm totals at three surrounding stations A, B and C were 107, 89 and 122 mm. The normal annual precipitation amounts of stations X, A, B and C are respectively, 978, 1120, 935 and 1200 mm. Estimate the storm precipitation for station X.*

Solution. N_x = Average annual precipitation at X = 978 mm.

$\therefore$ 10% of N_x = 97.8 mm.

Thus, maximum permissible annual ppt. at either of the three stations for taking ordinary mean = 978 + 97.8 mm = 1075.8 mm < 1120 and 1200 mm (given)

Hence, the annual ppt. at two of the three stations differ by more than 10% of N_x. Hence, weighted mean should be taken. Hence, using Eq. (7.6), we get

$$P_x = \frac{1}{3}\left[P_1 \frac{N_x}{N_1} + P_2 \frac{N_x}{N_2} + P_3 \frac{N_x}{N_3}\right]$$

or

$$P_x = \frac{1}{3}\left[P_a \frac{N_x}{N_a} + P_b \frac{N_x}{N_b} + P_c \frac{N_x}{N_c}\right]$$

Putting various given values, we get

$$P_x = \frac{1}{3}\left[107\left(\frac{978}{1120}\right) + 89\left(\frac{978}{935}\right) + 122\left(\frac{978}{1200}\right)\right]$$

$$= \frac{1}{3}[93.5 + 93.1 + 99.4] = \frac{1}{3}[286] = 95.3 \text{ mm}$$

Hence, the missing precipitation data = 95.3 mm **Ans.**

7.13. Checking the Consistency of Data of a Rain Gauge Station

Sometimes a significant change may occur in and around a particular rain gauge station. Such a change* occurring in a particular year, will start affecting the rain gauge data, being reported from that particular station. After a number of years, it may be felt that the data of that station is not giving consistent rainfall values. In order to detect any such inconsistency, and to correct and adjust the reported rainfall values, a technique, called **double mass curve method**, is generally adopted.

In this method, a group of 5 to 10 neighbouring stations are chosen, in the vicinity of the *doubtful station*. The yearly rainfall values, reported from this group of stations are serialled, and their mean yearly values are worked out for each consecutive year of available record. These mean yearly rainfall values (of the chosen group of stations) are serially arranged in a *reverse chronological order* (*i.e.* the latest year getting the first entry). Against these values, the recorded yearly rainfall values of the doubtful station are also serialled for each year. The cumulative values for both the columns are then worked out. The cumulative ppt. values of the doubtful station, X, say ΣP_x, and the cumulative values of the group averages say ΣP_{av} are then plotted on a graph paper, as shown in Fig. 7.16.

A decided change in the slope of the resulting curve (a st. line) persisting for more than 5 years, indicates a change in the precipitation regime of the station X.

The ppt. values at station X, prior to the period of change, are then corrected by using the relation :

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*Common significant changes that may occur are :

(*i*) Location of raingauge station may undergo a change, which may go unreported; (*ii*) the neighbouring area of the raingauge station may undergo significant changes, such as new heavy built up coming in and around the station ; (*iii*) surrounding ecosystem may undergo significant changes due to calamities like slides, forest fires etc. ; observational errors may creep in the reported data.

$$P_x' = P_x \frac{M'}{M} \qquad ...(7.7)$$

where P_x' = corrected ppt. at station X

P_x = original recorded ppt. at station X

M' = corrected slope of the double mass curve.

M = original slope of the double mass curve.

The method will become more clear on solving example 7.2.

Example 7.2. *The annual rainfall data, being reported from a station A, for 22 years are available, since 1969. In order to check the consistency of this data, six neighbouring stations have been chosen and the annual rainfall values of these stations have been averaged for all the years on record since 1969. These values are given in table 7.5.*

Table 7.5

Year	*Yearly ppt. at station A in cm*	*6 station average yearly ppt. in cm*
(1)	*(2)*	*(3)*
1969	*177*	*143*
1970	*144*	*132*
1971	*178*	*146*
1972	*162*	*147*
1973	*194*	*161*
1974	*168*	*155*
1975	*196*	*152*
1976	*144*	*117*
1977	*160*	*128*
1978	*196*	*193*
1979	*141*	*156*
1980	*158*	*164*
1981	*145*	*155*
1982	*132*	*143*
1983	*95*	*115*
1984	*148*	*135*
1985	*142*	*163*
1986	*140*	*135*
1987	*130*	*143*
1988	*137*	*130*
1989	*130*	*146*
1990	*163*	*161*

(a) Find out if any inconsistency in ppt. record of station A is indicated? And if yes, since when a change in the ppt. regime is indicated ?

(b) Adjust the recorded data at station A and determine its mean annual precipitation.

Solution. The given data is arranged in a reverse chronological order, and their cumulative values are worked out, in table 7.6.

Table 7.6

Year (1)	*Yearly ppt. at station A* P_a *in cm* (2)	*Cumulative yearly ppt. at A* ΣP_a *in cm* (3)	*6 station Av. yearly ppt.* P_{av} *in cm* (4)	*Cumulative 6 station Av. yearly ppt.* ΣP_{av} *in cm* (5)
1990	163	163	161	161
1989	130	293	146	307
1988	137	430	130	437
1987	130	560	143	580
1986	140	700	135	715
1985	142	842	163	878
1984	148	990	135	1013
1983	95	1085	115	1128
1982	132	1217	143	1271
1981	145	1362	155	1426
1980	158	1520	164	1590
1979	141	1661	156	1746
1978	196	1857	193	1939
1977	160	2017	128	2067
1976	144	2161	117	2184
1975	196	2357	152	2336
1974	168	2525	155	2491
1973	194	2719	161	2652
1972	162	2881	147	2799
1971	178	3059	146	2945
1970	144	3203	132	3077
1969	177	3380	143	3220

A graph is now plotted between the values of col. (3) and (5) as shown in Fig. 7.16. Corresponding years from Col. (1) are also marked on the corresponding plotted points, as shown in this figure.

A perusal of this figure shows that the inconsistency has occurred from 1978. Hence, the present data, since 1978 to 1990 will be treated as correct, and the previous data prior to the year 1978, will be corrected. In other words, the ppt. values of years 1977 to 1969 will be corrected.

The correction ratio is read out from this figure,

$$= \frac{M'}{M} = \frac{\text{Corrected slope of double mass curve}}{\text{Original slope of double mass curve}}$$

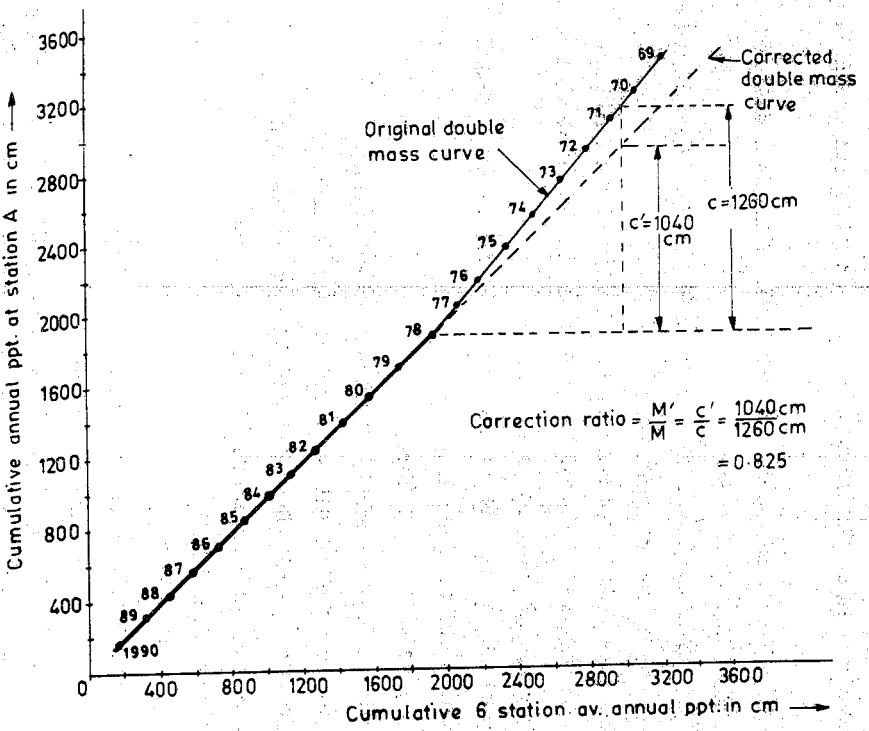


Fig. 7.16. Double mass curve plot for Example 7.2.

$$= \frac{C'}{C} = \frac{1040}{1260} = 0.825$$

The yearly ppt. values of station A (P_a) between the years 1977 to 1969 are thus, corrected by multiplying the original values by 0.825, as shown in table 7.7.

Table 7.7

Year	*Given ppt. values P_a in cm*	*Corrected ppt. values P_a' in cm* = 0.825 × *col.* (2)
(1)	*(2)*	*(3)*
1977	160	132
1976	144	119
1975	196	162
1974	168	139
1973	194	160
1972	162	134
1971	178	147
1970	144	119
1969	177	146

The final adjusted values of annual ppt. at station *A* are serialled in table 7.8, and their mean value is determined as equal to **141.6 cm. Ans.**

Table 7.8

Year	*Corrected/adjusted annual ppt. values at station A in cm*
1969	146
1970	119
1971	147
1972	134
1973	160
1974	139
1975	162
1976	119
1977	132
1978	196
1979	141
1980	158
1981	145
1982	132
1983	95
1984	148
1985	142
1986	140
1987	130
1988	137
1989	130
1990	163
	$\Sigma = 3115$
	$av. = \frac{3115}{22} = 141.6$ cm. **Ans.**

7.14. Design of Rain Gauge Network

Rainfall records constitute the most important and fundamental data required for hydrological investigations. They are required for : analysing storms, fixing design flood, forecasting floods in a river, reservoir regulation, etc. For all such studies and investigations, a well distributed network of rain gauge stations within the catchment is essential.

To obtain reliable results, the various rain gauges should be evenly and uniformly distributed within a given catchment. Moreover, the total number of rain gauges installed within a given catchment area should neither be too many as to be costly, nor should be too less as to give unreliable results. However, this **Areal density of rain gauges*** may vary considerably from region to region in any country. The density of gauges may also depend upon the basic purpose behind their installation.

For example, a relatively sparse network of stations (*i.e.* lesser number of gauges per unit area), would be sufficient for studies of large general storms or for determining annual average rainfall over large areas of large terrain. On the other hand, however, a

* The number of gauges per unit of area in the catchment.

dense network (*i.e.* more number of gauges per unit area) is required to determine the rainfall pattern in the thunder-storms.

If the network density is more (*i.e.* lesser area served per gauge), there is more probability that the storm centre will be recorded by the gauge. A network should also be planned to yield a representative picture of the areal distribution of rainfall. Hence, *there should be no concentration of gauges in heavy rainfall areas at the cost of dry areas, or vice-versa.* However, unfortunately, in actual practice, far flung and dry areas are generally left without gauges, because the cost of installing and maintaining a network, and accessibility of the gauge site to an observer are always very much taken into consideration, while deciding their locations.

In India, at present, there are about 5000 rain gauge stations spread over the vast area of about 31.5 lakh sq. km, which gives an average of about one station for every 630 sq km. This density is insufficient, and more gauges are required.

The minimum rain gauge density required for different types of catchments, as per the guidelines of World Meteorological Organisation, are given below :

(1) *For flat regions of temperate Mediterranean and Tropical zones* — one station per 600—900 sq. km. For countries with limited funds, population or other factors such as lack of communication, the density may be reduced to one station per 600—3000 sq. km.

(2) *For Mountainous regions of temperate Mediterranean and Tropical zones* — one station per 100—250 sq. km. For countries with problems mentioned above, the density may be reduced to one station per 250—1000 sq. km.

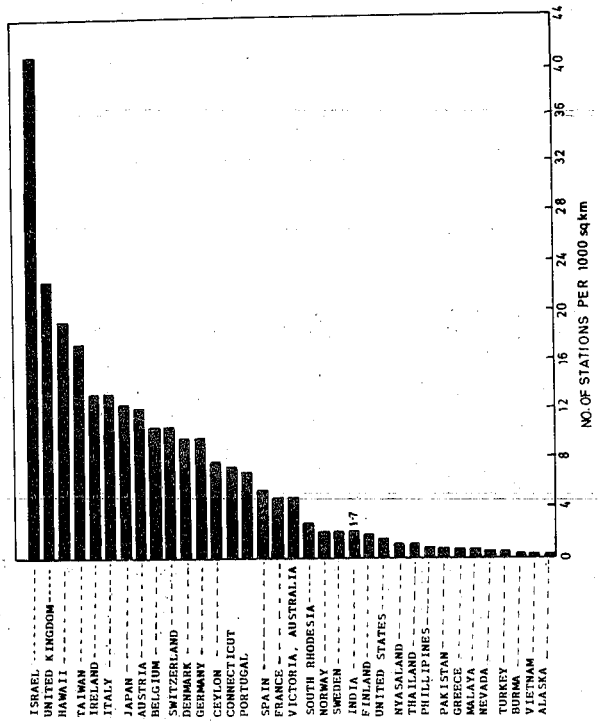


Fig. 7.17. Rain gauge density as adopted in different countries.

(3) *For arid or polar zones* — the minimum density may be one station per 1500—10,000 sq. km. depending upon the feasibility.

In India, however, the standard prescribed by a High Level Committee on floods set up by the Govt. of India, recommends to establish minimum 1 station per 500 sq. km. (*i.e.* 1 station per 200 sq. miles).

Fig. 7.17 represents the rain gauge density values, as adopted in different countries of the world. This figure reflects that Israel has the maximum rain gauge density of the order of 37 gauges/1000 sq. km of area, as against the Indian figure of 1.7 gauge per 1000 sq km area.

7.14.1. Optimum Number of Rain Gauges. Statistics has been used in determining the optimum number of rain gauges required to be installed in a given catchment. The basis behind such statistical calculations is : that a certain number of rain gauge stations are necessary to give average rainfall with a certain percentage of error. If the allowable percentage error is more, lesser number of gauges would be required. Based upon this statistical principle, the optimum number of rain gauges (N) can be obtained by the following equation :

$$N = \left(\frac{C_v}{E}\right)^2 \qquad ...(7.8)$$

where C_v = Coefficient of variation of rainfall based on the existing rain gauge stations; and

E = Allowable percentage error in the estimate of basic mean rainfall.

C_v can be computed as follows.

(*i*) Calculate mean average annual rainfall value as :

$$\overline{P} = \frac{\Sigma P}{n} \qquad ...(7.9)$$

where n is the number of rain gauges existing, and ΣP is the total rainfall, *i.e.* summation of their normal annual rainfall values.

(*ii*) Calculate mean of squares, *i.e.*

$$\overline{P^2} = \frac{\Sigma P^2}{n} \qquad ...(7.10)$$

(*iii*) Calculate Sample Standard Deviation, *i.e.*

$$\sigma = \sqrt{\frac{n}{n-1}\left[\overline{P^2} - (\overline{P})^2\right]} \qquad ...(7.11)$$

(*iv*) Calculate the coefficient of variation, *i.e.*

$$C_v = \frac{100\,\sigma}{\overline{P}} \qquad ...(7.12)$$

Example 7.3. *There are four rain gauge stations existing in the catchment of a river. The average annual rainfall values at these stations are 800, 620, 400 and 540 mm respectively. (a) Determine the optimum number of rain gauges in the catchment,*

if it is desired to limit the error in the mean value of rainfall in the catchment to 10%.
(b) How many more gauges will then be required to be installed.

Solution. (*i*) Mean rainfall $= \overline{P} = \frac{\Sigma P}{n} = \frac{800 + 620 + 400 + 540}{4} = \frac{2360}{4} = 590$ mm

(*ii*) $\Sigma P^2 = (800)^2 + (620)^2 + (400)^2 + (540)^2$

$= 6,40,000 + 3,84,400 + 1,60,000 + 2,91,600 = 14,76,000$

$\overline{P^2} = \frac{\Sigma P^2}{n} = \frac{14,76,000}{4} = 3,69,000$

(*iii*) Standard Deviation σ is given by

$$\sigma = \sqrt{\frac{n}{n-1}\left[\overline{P^2} - (\overline{P})^2\right]}$$

$$= \sqrt{\frac{4}{4-1}[3,69,000 - (590)^2]} = \sqrt{\frac{4}{3}[3,69,000 - 3,48,100]}$$

$$= \sqrt{27,867} = \mathbf{166.93}$$

(*iv*) The co-efficient of variation C_v is given by

$$C_v = \frac{100\,\sigma}{\overline{P}} = \frac{100 \times 166.93}{590} = 28.29$$

(*v*) Now, optimum number of rain gauges (N) is given by

$$N = \left(\frac{C_v}{E}\right)^2 \qquad \text{where } C_v = 28.29 \text{ (Calculated above)}$$

$E = 10\%$ (*i.e.* Per cent error allowed)

$$\therefore \qquad N = \left(\frac{28.29}{10}\right)^2 = \mathbf{8.004}\,; \quad \text{Say 8 Nos. } \mathbf{Ans.}$$

(*b*) Additional gauges required to be installed

$= 8 -$ Existing 4 gauges $= 8 - 4 = 4$ Nos. **Ans.**

7.15. Average Annual Rainfall and Index of Wetness

The amount of rain collected by a given rain gauge in 24 hours is known as daily rainfall, and the amount collected in one year is known as *annual rainfall.* This annual rainfall at a given station should be recorded over a number of years, say 35 to 40 years or so. In India, this rainfall cycle period is taken to be about 35 years. The mean of the annual rainfalls over a period of 35 years or so, is therefore, known as *average annual rainfall or normal annual rainfall* of the given station.

Whenever, we talk of the rainfall at any given place, we generally refer to the average annual rainfall of that place. Thus, when we speak that Cherrapunji gets 1270 cm of yearly rain, it means that this figure has been averaged over a long period of about 35 years. This is known as average rainfall. But in any given particular year, the rain may not fall equal to this amount. It may be less than this average value or may exceed this average value. The ratio of the actual rainfall in a given particular year at a given place to the normal rainfall of that place, is known as *index of wetness.*

Index of wetness

$$= \frac{\text{Actual rainfall in a given year at a given place}}{\text{Normal annual rainfall of that place}} \qquad \ldots(7.13)$$

Index of wetness, thus, gives us an idea of the wetness of the year, and hence, it indicates the deficiency of rain. 60% index of wetness means a rain deficiency of 40%. A deficiency of about 30—45% is known as large deficiency, 45 — 60% as serious deficiency, and more than 60% as disastrous deficiency.

The year in which the rainfall is less than the average annual rainfall, is called a bad or a sub-normal year; and if the rainfall in a given year is more than the average, then it is known as a good year. If the rainfall in a particular year is approximately equal to the annual average value, then it is known as a normal year or an average year.

7.16. Indian Rainfall

India is a tropical country, and its precipitation varies considerably in frequency, intensity, as well as in duration. The total precipitation occurs either as rainfall or as snowfall (in higher reaches). The largest amount of precipitation (75% or so) falls as rain during the *monsoons* (May—Oct.); whereas, moderate amount (25% or so) mainly as snow, falls during the winter season (Dec.—Feb.) as *winter precipitation.*

Besides these two major precipitation periods, *minor* quantity of rain falls, either as *post monsoon* (*i.e. retreating monsoon*) *rain* or as *pre-monsoon rain.*

Post monsoon rain (Oct.—Nov.) may fall in Tamil Nadu State and other nearby coastal areas; whereas, **Pre-monsoon rain** (March—May) may occur in Kerala, West Bengal, Assam, and other east coast areas.

7.16.1. Monsoon Precipitation. The *monsoon,* strictly speaking, the ***south-west monsoon*** (or *summer monsoon*), causes about 75% of annual precipitation, covering almost the entire country, except Jammu & Kashmir, and south eastern parts of South India (peninsula). Monsoon winds, infact, originate in the Indian Ocean, and thus appear first in Kerala, by the end of May. During such a summer season, south-westerly monsoon winds blow at speeds as high as 30 km/hr. These winds get divided into two branches, due to the relief of our country. It is the Indian peninsula, which rather deflects the monsoon, into the two branches, *viz.* :

(*i*) the Arabian Sea branch; and

(*ii*) the Bay of Bengal branch.

The Arabian Sea branch of monsoon, first of all, appears as early as middle or end of May, in Kerala. These moisture laiden monsoon winds strike the western ghats straight, thus causing heavy rain, of the order of 250 cm of so, between western ghats and west coast (Malabar coast). These winds, containing the residual moisture, then travel as far as east coast, causing smaller rain in Madras (30 cm or so). Another current of these winds move northwards, striking the hills of Chhota Nagpur, causing moderate to heavy rain there, of the order of 150 cm or so. A third part of these winds, moves over to Rajasthan, passing the parallel Aravali ranges without striking and without causing rain, till reaching and striking the hills of Punjab, causing moderate to heavy rain in Simla, Kangra, Dharamsala, etc. This Arabian Sea monsoon reaches Delhi by June end, causing low to moderate rain.

The *Bay of Bengal branch of monsoon* arises from the Bay of Bengal, and is deflected into India by the Arakan mountains of Burma, and by Himalayas.

A part of these monsoon winds move towards the Assam hills (Garo, Khasi and Jaintia). The winds, thus strike these high hills, causing very heavy rainfall here. So much so, that the place of maximum rainfall of India (*i.e.* Cherrapunji) is also situated here in Khasi hills. A part of these winds also advances westwards, towards Bihar and U.P., ultimately reaching Delhi by about June end *i.e. by* about the same time when the Arabian Sea branch reaches Delhi. But as the winds advance, they go on becoming less and less laiden with moisture, and hence the rainfall goes on reducing from Calcutta (160 cm) to Benares (100 cm) and to Delhi (60 cm).

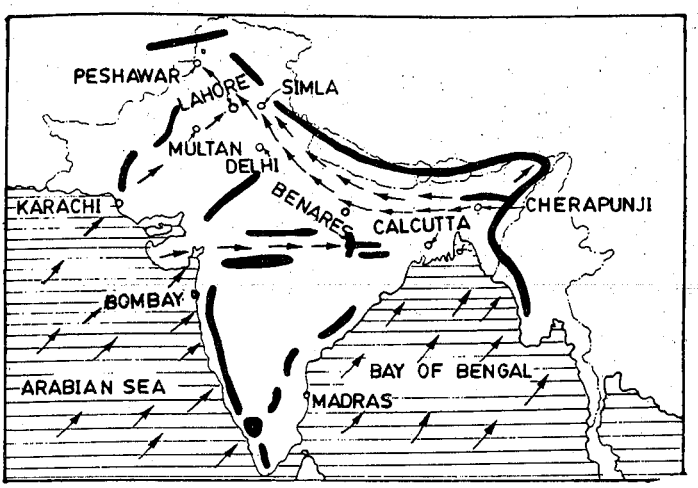


Fig. 7.18. Summer monsoons.

Infact, a low pressure region, called *monsoon trough*, is formed between these two monsoon branches, which extends from the Bay of Bengal to Rajasthan, and its position decides the precipitation pattern over the country. During the monsoon period, cloudy weather persists, giving relief from oppressive heat, and large amounts of rain falls in frequent spells almost over the entire country, owing to the passage of low pressure regions. Depressions formed in Bay of Bengal @ 2 to 3 Nos. per month, move along the trough, causing heavy rain up to as high an intensity as 20 cm/day or so. The heaviest rain falls in Assam and north-eastern States with annual value being of the order of 200—400 cm. West coasts and Western ghats (200—300 cm), West Bengal (120—160 cm), U.P., Haryana and Punjab (100—120 cm), then receive decreasing amounts of rain.

7.16.2. Winter Precipitation. By about mid December, *western disturbances* of extra tropical origin travel eastward to India, crossing Afghanistan and Pakistan, causing heavy rainfall and snowfall in Himalayas and Jammu & Kashmir. Some light rainfall also occurs in northern plains.

Lighter rains (10—12 cm) during winter are also caused in southern parts of Tamil Nadu, by *winter monsoon* (*north-east monsoon*), passing over Bay of Bengal and striking the eastern ghats.

Standard charts, showing the Average annual rainfall and normal Monsoon rainfall values all over India, are given in author's another book titled "Hydrology and Water Resources Engineering".

7.17. The Mean Rainfall Over a Drainage Basin

A given drainage basin is divided into various parts or sub- basins, and rain gauge stations are evenly distributed over that basin. The rain-catch at one station in a basin may be different from that of a second station in the same basin. An average value of these rain-catches is worked out, so as to get an idea of the average precipitation on the entire basin. The following methods are generally used to work out the mean rainfall on an area.

7.17.1. Arithmetical Mean Method. This is the simplest method that can be used. It consists of averaging all the amounts that have been recorded at the various stations in the area, as given below :

$$P = \frac{P_1 + P_2 + P_3 + P_4 + \ldots + P_n}{n} \qquad \ldots(7.14)$$

where P = the mean precipitation on the basin. P_1, P_2, P_3, ..., P_n are respective precipitations at stations 1, 2, 3, ..., n ; and n is the total number of stations.

If the stations are uniformly distributed over the area, and the rainfall rate does not differ much at various stations, then this method is quite satisfactory. But if there are many topographic differences, and the rainfall rates vary considerably, then this method may lead to large errors.

7.17.2. Thiessen's Mean Method. In this method, adjacent stations are joined by straight lines, thus dividing the entire area into a series of triangles. Perpendicular bisectors are erected on each of these lines, thus forming a series of polygons, each containing one and only one rainfall station. It is assumed that the entire area within any polygon is nearer to the rainfall station that is included in the polygon than to any other rainfall station. The rainfall recorded at that station is, therefore, assigned to that polygon. If P is the mean rainfall on the basin, and area of the basin is A, then

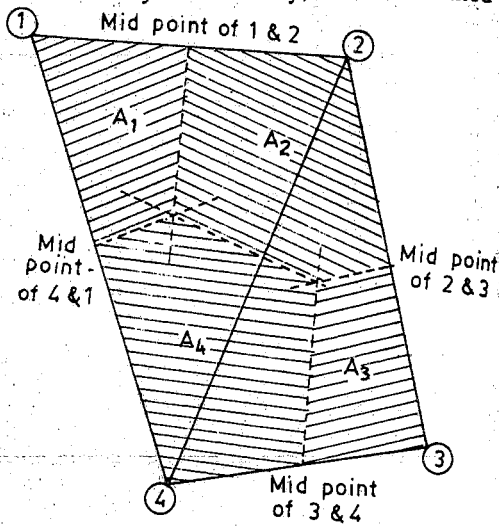


Fig. 7.19. Thiessen's Polygon method.

$$P = \frac{A_1P_1 + A_2P_2 + A_3P_3 + \ldots + A_nP_n}{A} \qquad \ldots(7.15)$$

where P_1, P_2, P_3, ..., P_n represent rainfalls at the respective stations, whose surrounding polygons have the areas A_1, A_2, A_3, ..., A_n respectively.

A graphical representation of this method is shown in Fig. 7.19 by taking 4 stations.

To divide the area into triangles, most frequently the shorter of the two diagonals is to be joined. Sometimes, the reverse is to be adopted when the clear polygons are not formed and they overlap.

So in Fig. 7.19, let us join the various stations namely 1, 2, 3 and 4. And then divide it into triangles by joining the shorter diagonal, *i.e.* 2 and 4, then draw perpendicular bisectors (shown by dotted lines) so as to divide the area into polygons. The figure is self-explanatory.

The method will become more clear on solving the following numerical example.

Example 7.4. *Find the mean precipitation for the area sketched in Fig. 7.20 by Thiessen's method. The area is composed of a square plus an equilateral triangular plot of side 4 kilometres. Rainfall readings in centimetres at the various stations are also given in the Fig. 7.20.*

(U.P.S.C., Engg. Services, 1968)

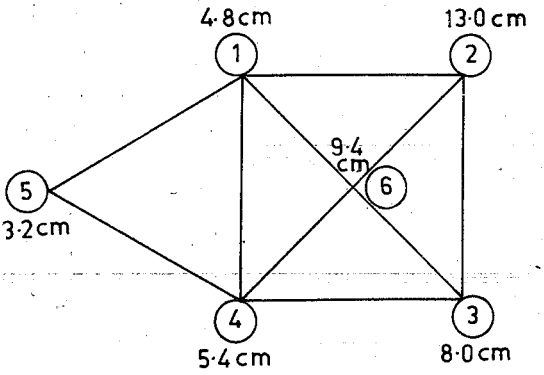


Fig. 7.20

Solution. First of all, divide the entire area into triangles by joining the various consecutive stations, *i.e.* 1, 2, 3, 4, 5, and 1, 6, 3, and 1, 4. Now, draw perpendicular bisectors on these lines to divide the area into polygons, as shown in Fig. 7.21.

In triangles *ABC* and *DBE*

Side *DE* is the join of the mid-points of *AB* and *BC*

Therefore $DE = \frac{1}{2} AC$

If a = side of the square

then $AC = \sqrt{2} \cdot a$

$\therefore \quad DE = \frac{\sqrt{2} \cdot a}{2} = \frac{a}{\sqrt{2}}$

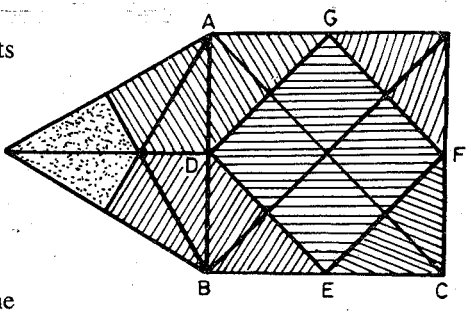


Fig. 7.21.

Area of the square *DEFG* feeding the station No. 6

$$= \frac{a}{\sqrt{2}} \times \frac{a}{\sqrt{2}} = \frac{a^2}{2} \qquad ...(1)$$

Area of square plot contributing to each corner station

$$= \frac{1}{2} \cdot \frac{a}{2} \cdot \frac{a}{2} = \frac{a^2}{8}$$

In equilateral triangle, the three vertices stations will be fed equally, *i.e.* equal to 1/3 × area of triangle

$$= \frac{1}{3} \times \left(\frac{1}{2} a \cdot \frac{\sqrt{3}}{2} a \right) = \frac{1}{3} \times \frac{\sqrt{3}}{4} a^2 = \frac{a^2}{4\sqrt{3}}$$

Thus, the areas feeding various stations can now be stipulated as below :

A_1 = Area due to triangular plot

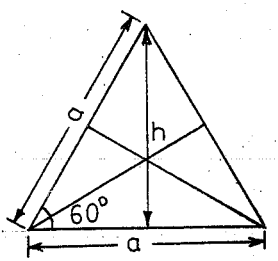
+ Area due to square plot

$$= \frac{a^2}{4\sqrt{3}} + \frac{a^2}{8} \text{ ; where } a = 4 \text{ km}$$

$$= 0.26925\, a^2 = 0.26925 \times 16 = 4.308 \text{ km}^2$$

$$A_2 = \frac{a^2}{8} = \frac{16}{8} = 2 \text{ km}^2$$

$$A_3 = \frac{a^2}{8} = 2 \text{ km}^2$$



$\frac{h}{a/2} = \tan 60° = \sqrt{3}$; $h = \frac{\sqrt{3}}{2} a$

Fig. 7.22.

$$A_4 = A_1 = 4.308 \text{ km}^2$$

$$A_5 = \frac{a^2}{4\sqrt{3}} = 2.309 \text{ km}^2$$

$$A_6 = \frac{a^2}{2} = 8 \text{ km}^2$$

Using $P = \dfrac{P_1A_1 + P_2A_2 + \ldots + P_nA_n}{A}$, we have

$$P = \frac{4.8 \times 4.308 + 13 \times 2 + 8 \times 2 + 5.4 \times 4.308 + 3.2 \times 2.309 + 9.4 \times 8}{4.308 + 2 + 2 + 4.308 + 2.309 + 8} = 7.36 \text{ cm}$$

Hence, the mean precipitation over the area = 7.36 cm. **Ans.**

7.17.3. The Isohyetal Method. Isohyets are the contours of equal rainfall. They are drawn on the map by using common sense, after the rainfall at each station is plotted.

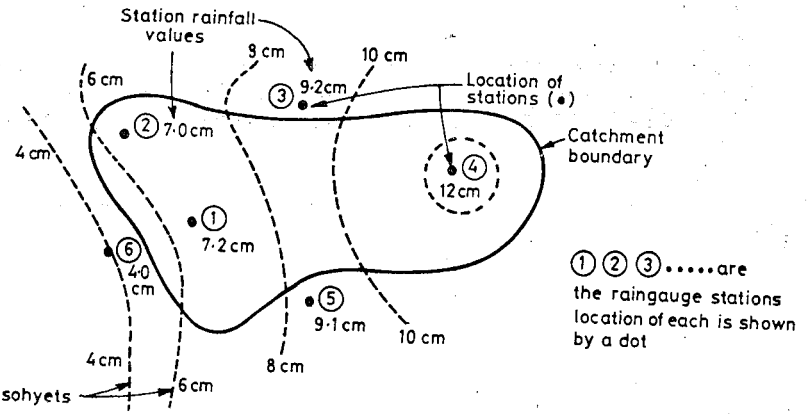


Fig. 7.23. Isohyetal method for computing average rainfall.

The area between the adjacent isohyets is either estimated on the graph paper or measured by planimeter. Let them be $A_1, A_2, A_3, \ldots, A_n$; and let the average precipitation for these areas be $P_1, P_2, P_3, \ldots, P_n$; then the mean precipitation P on the basin is given by

$$P = \frac{P_1A_1 + P_2A_2 + P_3A_3 + \ldots + P_n \cdot A_n}{A_1 + A_2 + A_3 + \ldots + A_n} \qquad \ldots(7.16)$$

If A_1 is the area between the contours of 6 cm and 8 cm (Fig. 7.23).

then $P_1 = 7$ cm

Similarly, here in figure, we have

$P_2 = 9$ cm

Note : Since the area between the two adjacent Isohyets is assumed to be having a rainfall equal to the arithmetic average of rainfall values represented by the two isohyets, the method evidently assumes uniform arial distribution of rainfall in between two isohyets. Such an assumption may not always be true, as orological considerations of the ground below might affect this assumption.

Example 7.5. *The isohyets for annual rainfall over a catchment basin were drawn. The areas of strips between the isohyets are indicated below in table 7.9. Find the average depth of annual precipitation over the basin.*

Table 7.9.

Isohyets cm	*Area in sq. km.*	*Isohyets cm*	*Area in sq. km.*
75—85	*580*	*105—115*	*1,000*
85—95	*2,960*	*115—135*	*610*
95—105	*2,850*	*135—155*	*160*

(U.P.S.C. Engg. Services, 1970)

Solution. $$P = \frac{580\times 80 + 2960\times 90 + 2850\times 100 + 1000\times 110 + 610\times 125 + 160\times 145}{580 + 2960 + 2850 + 1000 + 610 + 160}$$

$$= \frac{46{,}400 + 2{,}66{,}400 + 2{,}85{,}000 + 1{,}10{,}000 + 76{,}250 + 23{,}200}{8{,}160}$$

$$= \frac{8{,}08{,}250}{8{,}160} = \mathbf{99.05\ cm}\quad \textbf{Ans.}$$

7.17.4. Merits and Demerits of Three Methods. As is quite evident, the 1st, and 2nd methods, *i.e.* 'arithmetic mean' and 'Thiessen's mean' are only mechanical and mathematical processes and do not require any special skill, but on the contrary, the third method, *i.e.* 'the Isohyetal method' requires a lot of extra special judgement for drawing the contours' (isohyets). The third method is the most accurate method, if the completion of contours (isohyets) is best done. But the accuracy depends upon the judgement of the person completing the contours.

The chief advantage possessed by the 'Thiessen's' and 'Isohyetal' methods over the 'arithmetic mean method' is the fact that in these methods, suitable importance is attached to the various stations, which is not so in 'arithmetic mean' method. Also in these methods, the stations which are situated beyond the boundary of the drainage basin, are also used to determine the mean rainfall on the basin, but their influence reduces as their distance from the boundary increases.

Example 7.6. *The isohyetal map for 24 hour storm gave the areas enclosed between different isohyets, as follows :*

Isohyets in mm	*21*	*20*	*19*	*18*	*17*	*16*	*15*	*14*	*13*	*12*
Enclosed area in sq. km.	*543*	*1345*	*2030*	*2545*	*2955*	*3280*	*3535*	*3710*	*3880*	*3915*

Determine the average depth of rainfall over the basin.

Solution. The computations for calculating average rainfall by using the formula $P = \frac{P_1A_1 + P_2A_2 + \ldots}{A_1 + A_2 + \ldots}$ are easily done in a tabular form, as shown in Table 7.10.

Table 7.10

Isohyet mm (1)	*Area between each isohyet and basin boundary in km²* (2)	*Net incremental area between two isohyets i.e. A_1, A_2, A_3, ... in km²* (3)	*Av. rainfall on the incremental areas i.e. av. of two isohyets; i.e., P_1, P_2, P_3, ... in mm* (4)	*Rainfall volume on incremental areas;, i.e., P_1A_1, P_2A_2, P_3A_3, ... col. (3) × col. (4) in mm. km²* (5)
21	543	543*	21**	11,403
20	1345	802	20.5	16,441
19	2030	685	19.5	13,357
18	2545	515	18.5	9,527
17	2955	410	17.5	7,175
16	3280	325	16.5	5,362
15	3535	255	15.5	3,952
14	3710	175	14.5	2,537
13	3880	170	13.5	2,295
12	3915	35	12.5	437
	Σ	3915		72,486

$$\text{Av. Ppt over the basin} = \frac{\Sigma PA}{\Sigma A} = \frac{72486}{3915}\ \text{mm} = \mathbf{18.51\ mm.}\quad \textbf{Ans.}$$

*for the end contour, this is the area between the end isohyet and the basin boundary.

**The average value being the same as the value of the end isohyet.

ANALYSING PRECIPITATION DATA

7.18. Characteristics of a Rain-storm

A rainfall at a place can be completely described, if its intensity, duration, and frequency, are known. The intensity of a rain is the rate at which it is falling; the duration is the time for which it is falling with that given intensity, and frequency is the number of times it falls.

7.18.1. Rain Hyetograph. The **intensity of a rain** is expressed in cm/hr, but this rate at which the rain falls, changes continuously throughout the storm. It may rain 5 cm in one hour giving an average rainfall rate of 5 cm/hr. However, during this particular hour, sometimes the rainfall rate will greatly exceed 5 cm/hr, and sometimes it may be much less than 5 cm/hr. An idealized curve showing the intensity variation with time is shown in Fig. 7.24 (*a*), and is known as the *time-intensity pattern.*

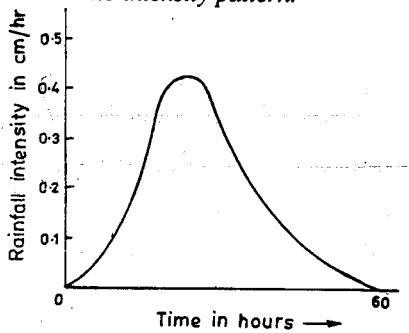


Fig. 7.24 (*a*) Time intensity pattern

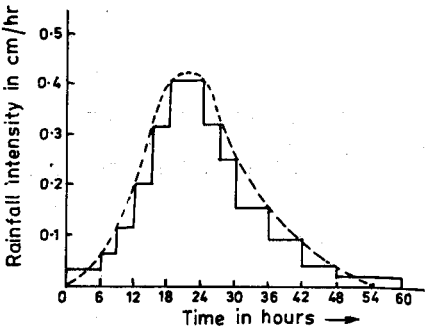


Fig. 7.24 (*b*) Rain hyetograph.

The above time-intensity curve is generally assumed to be made up of straight lines, as shown in Fig. 7.24 (*b*), and is called a *rain hyetograph.* A **hyetograph** can, therefore, be defined as a plot of intensity of rainfall against the time interval, represented as a bar chart.

The area under a hyetograph represents the total rainfall received in that period. This chart is very useful in representing the characteristics of a storm, and is particularly important in developing the design storms to predict extreme floods.

The time interval used, depends on the purpose; say for example, in urban drainage problems, small durations are used ; while in flood flow computations in larger catchments, the intervals are of about 6 hour.

7.18.2. Mass Curve of Rainfall. If the total accumulated precipitation is plotted against time, the curve obtained is known as the *mass curve of the storm*, and is shown in Fig. 7.25. The curve rises steeply in the beginning and then tends to become constant.

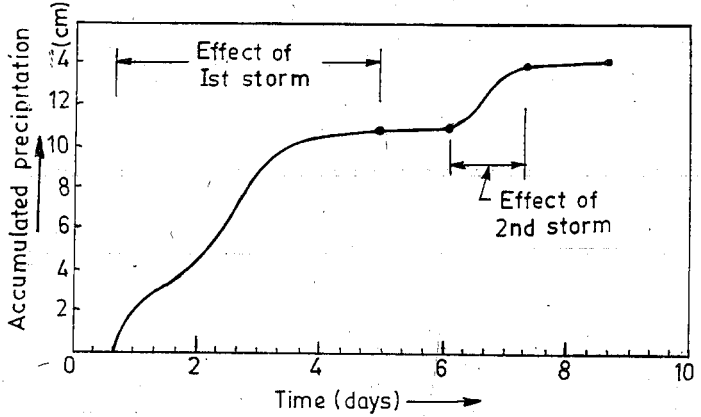


Fig. 7.25. Mass curve of rainfall.

MAXIMUM PRECIPITATION

7.18.3. Moving Average Curve of Annual Rainfalls. The rainfall records of various years can be graphically represented by plotting the annual rainfall values on *Y*-axis and the respective years on *X*-axis, as to get a bar graph, as shown in Fig. 7.26. Such a bar graph, however, does not reflect any clear trends or patterns in the rainfall due to abrupt variations in individual years. In order to suppress these and to bring out the general trend of the rainfall over say 3 years or 5 years, the averages of the three (or five) consecutive years, are computed progressively, and are then plotted, as shown in Fig. 7.26.

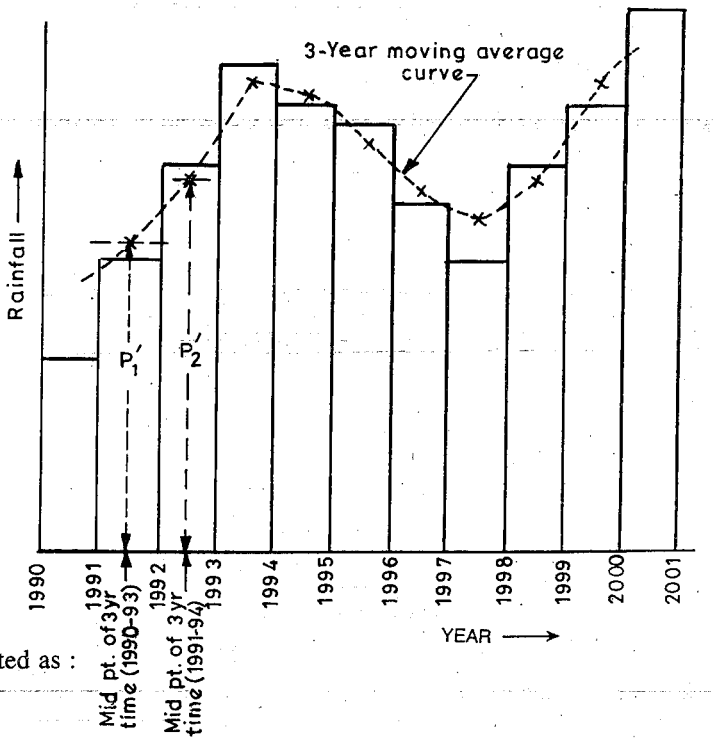


Fig. 7.26. Moving average curve of annual precipitation.

Thus, for a **three year moving mean**, the various values of rainfall to be plotted against time (mean of 3 years) shall be calculated as :

$$P_1' = \frac{P_1 + P_2 + P_3}{3} \qquad P_2' = \frac{P_2 + P_3 + P_4}{3} \qquad P_3' = \frac{P_3 + P_4 + P_5}{3}$$

The values of P_1', P_2', P_3' ..... are then plotted against the corresponding times (mid pts of respective 3 years). The plot so obtained will be known as the *3 year moving average curve of annual rainfall*, and will indicate the *general trends regarding increase or decrease of annual precipitation with time.*

7.18.4. Depth-Area-Duration (DAD) Relationship. The **areal characteristics of a storm** of a given duration is reflected in its depth-area relationship. Such a relation is represented by a curve of the type shown in Fig. 7.27. *Different such curves will be obtained for different storms of a given duration*, as shown in Fig. 7.28; from which maximum observed DAD curve of a particular duration can be determined.

A depth-area duration curve expresses graphically the relation between progressively decreasing average depth of rainfall over a progressively increasing area (from the centre of the storm outward to its edges) for a given duration of rainfall.

The immediate purpose of DAD analysis of a particular storm is to determine the largest average depth of rainfall that fell over various sizes of area during the standard passage of time in hours or days; such as the largest average depth of rainfall falling over 500 sq. km in 1 day. Such values when determined for each of the past storms occurring on the area, as well as for each of the *transposable storms**, provide the basic data to estimate the **probable maximum precipitation (PMP)** for the basin in question.

A *depth-area curve* can be constructed from any isohyetal map of rainfall of the given duration, by measuring the areas associated with each isohyet of rainfall depth, and then plotting the progressively decreasing average depth on *Y*-axis, and the corresponding areas on *X*-axis. Different such curves can be plotted for different duration storms, to obtain a composite DAD chart, for a common areal base. Incremental isohyetal maps for storms of selected durations, such as 6 hr. 12 hr. 1 day, 2 day, 3 day, etc. are required in this procedure, which, will become more clear when we solve Example 7.7.

Example 7.7. *Estimate the average depth of precipitation (from depth area curve) that may be expected over an area of 2400 sq. km, due to the storm of 27th Sept. 1978, lasting for 24 hours, assuming the storm centre to be located at the centre of the area. The isohyetal map for the storm gave the areas enclosed between different isohyets as follows :*

Isohyets in mm	*21*	*20*	*19*	*18*	*17*	*16*	*15*	*14*	*13*	*12*
Enclosed area (in sq. km.)	*543*	*1345*	*2030*	*2545*	*2955*	*3280*	*3535*	*3710*	*3880*	*3915*

(U.P.S.C. Engg. Services, 1986)

Solution. In order to plot the depth-area-duration curve, we have to, first of all, analyse the given data to determine the **equivalent-uniform depths** (EUD)** of rainfall, as computed in column (7) of Table 7.11, which is otherwise self-explanatory.

Table 7.11

Isohyet (given) mm	*Area enclosed between isohyet and basin boundary in km²*	*Net incremental area between isohyetes km²*	*Av. rainfall on incremental areas mm*	*Rainfall volume on incremental areas (mm.km²) i.e. col. (3) × (4)*	*Cumulative Volume (mm.km²) Σ (5)*	*Equivalent Uniform Depths E.U.D. (mm) Col. (6) / Col. (2)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
21	543	543	21	11,403	11,403	21.00
20	1345	802	20.5	16,441	27,844	20.70
19	2030	685	19.5	13,357	41,201	20.30
18	2545	515	18.5	9,527	50,728	19.93
17	2955	410	17.5	7,175	57,903	19.59
16	3280	325	16.5	5,362	63,265	19.29
15	3535	255	15.5	3,952	67,217	19.01
14	3710	175	14.5	2,537	69,754	18.80
13	3880	170	13.5	2,295	72,049	18.57
12	3915	35	12.5	437	72,486	18.51

* A storm occurring on a similar basin, when considered to be equivalent to a likely storm that can occur on the basin under consideration, is called a transposable storm. Storm transposition, therefore, means the application of a storm, from one area to another area, within the same meteorological homogeneous region.

** mean depths of rainfall over increasing areas of a basin

A curve is then plotted between equivalent uniform depths (*EUD*) of column (7), and the areas enclosed of column (2), as shown in Fig. 7.27.

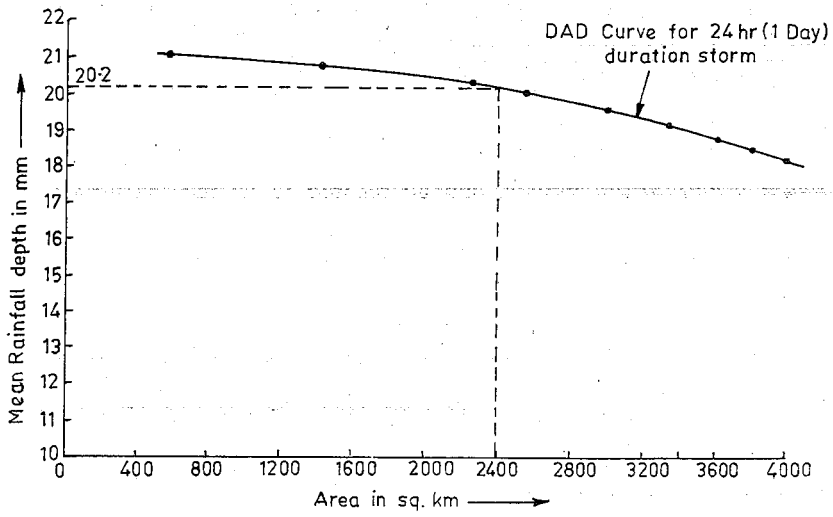


Fig. 7.27. DAD curve of 1 day rainfall for example 7.7.

From this curve, the average depth of precipitation for 2400 sq. km area is read out as 20.2 mm. **Ans.**

7.18.4.1. Equation of depth area-duration curve. The depth area curve for a given rain of a particular duration is of the type shown in Fig. 7.27. Such a curve can be expressed in a mathematical form, as :

$$P = P_0 (e)^{-K/A^n} \qquad ...(7.17)$$

where P = average depth in *cm* over an area A km^2

P_0 = Highest amount of rainfall in cm at the storm centre

K and n = constants for a given region.

On the basis of the study of a number of severest storms in North India, Dhar and Bhattacharya (1975) have computed the following values of constants K and n for storms of different durations.

Table 7.12

Duration in day	*K*	*n*
1	0.0008526	0.6614
2	0.0009877	0.6306
3	0.001745	0.5961

Since the storm centre may not coincide with a rain-gauge station, the value of P_0 may not be exactly known. Hence, in the analysis of large area storms, the highest station rainfall is taken as the average depth (P) over an area of 25 km^2, to workout P_0. The above equation (7.17) is useful in extrapolating an existing storm data over an area.

7.18.4.2. Maximised depth-area-duration curves. The depth-area curve (for a given duration) can be worked out, as explained above, for any given storm. All severer storms can be used to compute such curves, and the curve which gives *maximum precipitation values*, can be refered to as the **'observed maximum DAD curve'** of a particular duration (Refer Fig. 7.28 (*a*); DAD curve at the top is the observed maximum 3 day curve for Tajewala catchment). Such maximum observed precipitation curves of different durations are usually considered to project the *standard project storms* of different durations. These values are, however, further enhanced for worst and rare possible combinations of rain causing hydrometeorological factors, to obtain *absolute maximum DAD curves*, called *maximised DAD curves*, which represent the *limiting estimated values, or probable maximum values.*

Such maximised DAD curves at various durations are plotted and kept ready in the design office for subsequent use. A set of such **maximised DAD curves** for Damodar basin is shown in Fig. 7.28 (*b*).

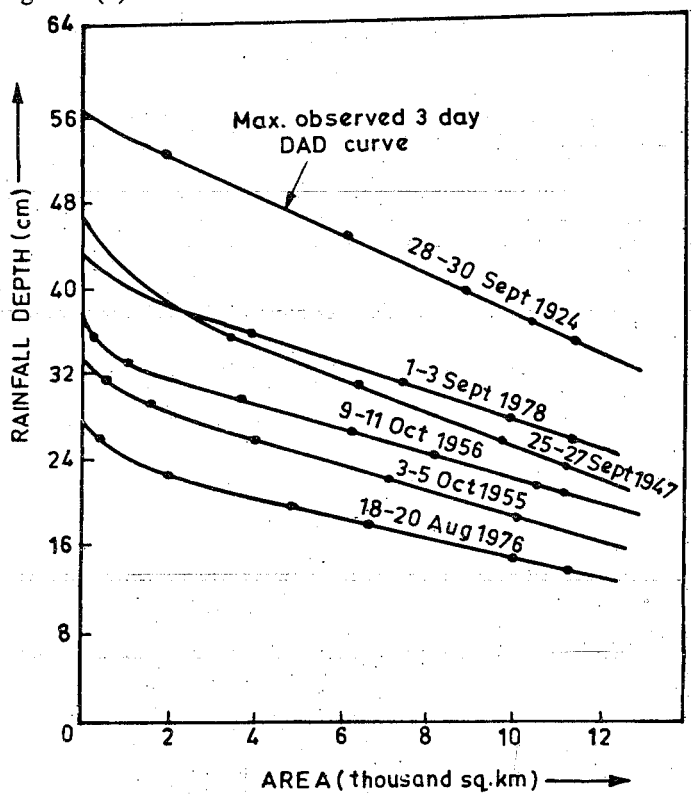


Fig. 7.28 (*a*). DAD curves of 3 day duration storms for Yamuna catchment upto Tajewala.

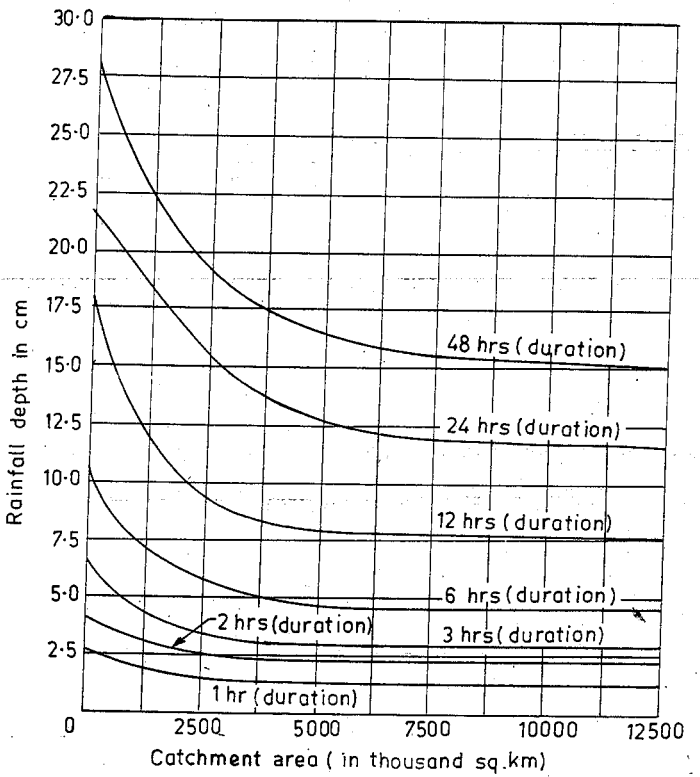


Fig. 7.28 (*b*). Typical maximised DAD curves for Damodar Basin (as supplied by CWC).

7.18.5. Frequency of a Storm or Recurrence Interval of a Storm. By recurrence interval (R.I.) of a given storm is meant the time interval during which the given storm is likely to be equalled or exceeded. Thus, if we say that at a given station, the maximum precipitation of 20 cm has got a recurrence interval of 5 years, we mean that on this station, the chances of rainfall are such that once in five years, rain is likely to equal or exceed 20 cm. This does not mean that a rain equal to 20 cm or more, will occur after every five years. It may occur twice in one set of five years and may not occur at all for consecutive eight years or so. What it means is, that during 100 years or so, such a rain will occur 20 times or so.

7.18.5.1. California probability method to determine recurrence interval. (T) If N is the total number of years of record, and T is the recurrence interval, then it can be easily stated that

$$N = T \cdot m \qquad \text{...(7.18)}$$

where m is the number of times the given rain is equalled or exceeded, and is known as the **ranking of the storm** or *ranking number.*

As an example, suppose, the rainfall records at a given station have been kept for 52 years, and it is desired to pick up the storm that is expected to appear once in eight

years; then $T = 8$, $N = 52$, and we have $m = \frac{52}{8} = 6.5$. Since the ranking of the storm is 6.5, we would pick up the 6th most severe storm. To make the things very clear, we solve the following example.

Example 7.8. *The values of annual precipitation at a rain-gauge station expressed in cm per year in chronological sequence from 1956 to 1967 are indicated below :*

36.5	*29.0*	*56.2*	*82.0*	*27.8*	*23.4*
71.2	*48.3*	*31.4*	*18.1*	*29.0*	*65.6*

Estimate the value of precipitation which has a recurrence interval of 5 years, using the probability or statistical method. (*U.P.S.C. Engg. Services, 1969*)

Solution. Total No. of years on record (N) = 12 years

Recurrence interval (T) = 5 years.

$\therefore$ m = Ranking of the storm $= \frac{N}{T} = \frac{12}{5} = 2.4$

Therefore, we should pick up the 2nd severest storm, *i.e.* 71.2 cm. (First most severe storm is 82.0 cm, and next severe storm is 71.2 cm).

Hence, the value of precipitation, which has a recurrence interval of five years, is 71.2 cm. **Ans.**

Limitations of Simple Probability Method. The recurrence interval or frequency can be determined easily by this simple probability method. However, a difficulty arises in determining the recurrence interval of very severe storms. Because, if there is a record of 40 years at a place, then the most severe storm will have its ranking equal to 1, and therefore, $T = \frac{N}{m} = \frac{40}{1} = 40$. The recurrence interval for this storm will come out to be 40 years. Moreover, this value is based on only one sample of 40 years of records, if no such strong rain equal to or more than this, falls during the next 40 years; then the frequency would be calculated at 80 years. On the other hand, if two such strong rains fall during the next 40 years, then the frequency would be calculated as $\frac{80}{3} = 26.7$ years.

Thus, it can be concluded that the frequency of the largest rain as determined from the equation $N = T.m$, cannot be relied upon, and it should be considered only as a very rough & approximate value. Similarly, the frequency of the next severest storm as calculated from the equation $N = T.m$, will also be approximate, but the approximation in this case will be lesser than that in the first case. Hence, for severe rains, the method gives only approximate values. It is, thus, very much possible, to calculate for a given storm, a frequency of 2 years from a ten years record, and to calculate 5 years frequency for the same storm on the basis of 60 years of records.

Secondly, if we want to determine a storm having recurrence interval of more than the total number of years of which the records are available, it is not possible to find it out by using the simple probability method *i.e.* $N = T.m$ (Because, in that case, T is greater than N, and

therefore, m is less than one, *i.e.* ranking of storm is less than 1, which is not known). These are the two *limitations of this method.*

In order to make use of the above method to determine the storm of such a high recurrence interval, the available records are sometimes plotted graphically. The curve is extended back, so as to determine the storm of the desired recurrence interval.

In this method, a graph is plotted between the recurrence interval (R.I.) on X-axis and precipitation in cm on Y-axis, as shown in Fig. 7.29.

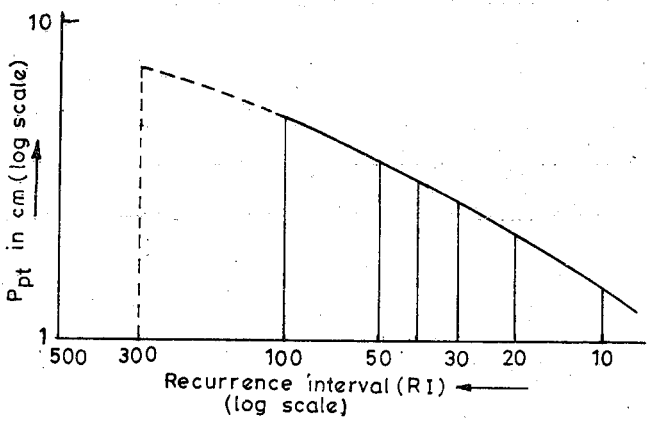


Fig. 7.29. Extrapolating the recurrence interval.

The curve can be extended back, so as to determine the precipitation of a storm having a higher recurrence interval. The method can be easily understood on solving the following example.

Example 7.9. *The maximum values of 24 hours summer precipitation at a rain gauge station expressed in cm from 1940 to 1969 are indicated below :*

10.7	*11.2*	*10.8*	*9.6*	*14.9*	*15.2*	*12.0*	*12.2*	*15.8*	*16.8*
9.7	*11.3*	*11.6*	*11.9*	*12.4*	*12.7*	*6.4*	*10.5*	*9.2*	*18.7*
17.1	*17.7*	*16.9*	*15.4*	*13.8*	*12.9*	*16.3*	*15.7*	*16.6*	*17.2*

Estimate the maximum precipitation having a recurrence interval of (a) 5 years (b) 10 years (c) 20 years (d) 40 years.

Solution. Let us first arrange the values of precipitation in the decreasing order with their corresponding years, as shown in Table 7.13. Value of recurrence interval (T) are then calculated in col. (4) of this table, which is otherwise self explanatory.

Table 7.13

Year	*Precipitation in cm*	*Ranking of storm m*	*Recurrence interval in years* $T = \frac{N}{m} = \frac{30}{m} = \frac{30}{(3)}$
(1)	*(2)*	*(3)*	*(4)*
1959	18.7	1	30.0
1961	17.7	2	15.0
1969	17.2	3	10.0
1960	17.1	4	7.5
1962	16.9	5	6.0
1949	16.8	6	5.0
1968	16.6	7	4.3
1966	16.3	8	3.8
1948	15.8	9	3.3
1967	15.7	10	3.0
1963	15.4	11	2.7
1945	15.2	12	2.5
1944	14.9	13	2.3
1964	13.8	14	2.1
1965	12.9	15	2.0
1955	12.7	16	1.9
1954	12.4	17	1.8
1947	12.2	18	1.7
1946	12.0	19	1.6
1953	11.9	20	1.5
1952	11.6	21	1.4
1951	11.3	22	1.4
1941	11.2	23	1.30
1942	10.8	24	1.25
1940	10.7	25	1.20
1957	10.5	26	1.15
1950	9.7	27	1.10
1943	9.6	28	1.07
1958	9.2	29	1.03
1956	6.4	30	1.00

Note. Column 4 of Table 7.13, shows the values of R.I. for all precipitations. Hence, the amount of ppt. for any R.I. can be directly read from this Table.

Case (*a*) When the recurrence interval or frequency is 5 years, $T = 5$ years, and the total number of years for which the records are available $N = 30$ years. Since $N = T.m$, therefore, $30 = 5 \times m$, or $m = 6$, *i.e.* ranking of the required storm = 6; so choose the 6th severest storm = 16.8 cm. **Ans.**

Case (*b*) When $T = 10$ years, $N = T.m$, therefore $30 = 10 \cdot m$ or $m = 3$, so choose the third severest storm = 17.2 cm. **Ans.**

Case (*c*) When $T = 20$ years, $N = T.m$, therefore, $30 = 20 \cdot m$, or $m = 1.5$; hence choose the first severest storm, *i.e.* 18.7 cm. **Ans.**

Case (*d*) When $T = 40$ years, $N = T.m$, therefore, $30 = 40 \cdot m$. Hence $m = \frac{30}{40} = 0.75$.

The storm ranking does not lie within the available records. So let us plot a graph between frequency (or recurrence interval) and maximum precipitation on log-log graph paper, as shown in Fig. 7.30. Then extend the curve, so as to determine the maximum

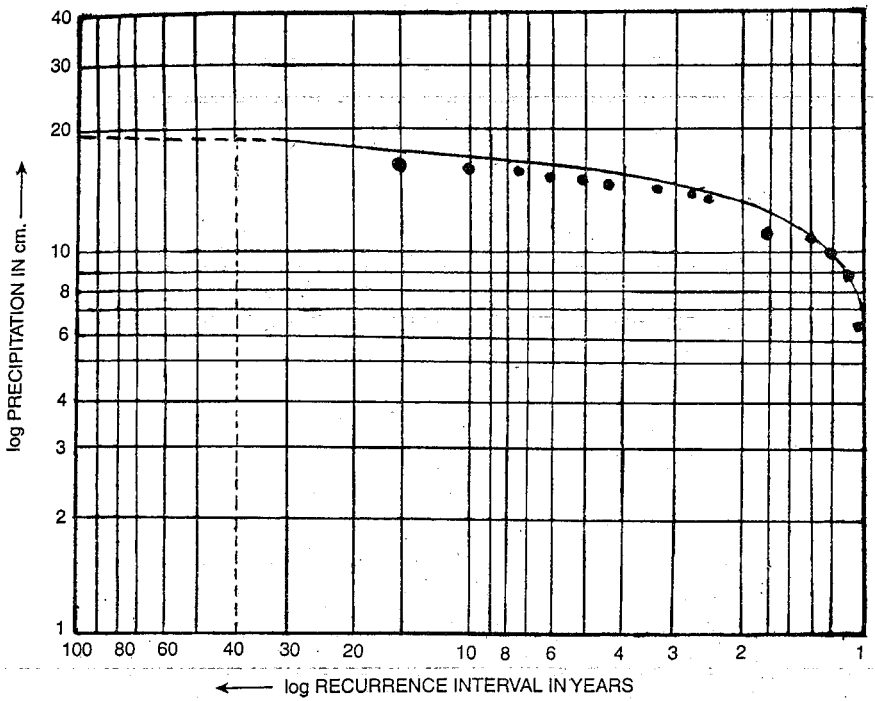


Fig. 7.30

precipitation as equal to 19.5 cm (from graph) for a recurrence interval of 40 years, as shown in Fig. 7.30 . **Ans.**

7.18.5.2. Station year method to determine the recurrence interval. Station year method is used to determine recurrence intervals which are longer, and hence, cannot be computed from the available records of a single station. This method is an extension of the method $N = T.m$, (described earlier) with the only difference that the records of various stations are combined together. All the records which are available from the various stations situated in a *meteorologically homogeneous* area are combined together, so as to represent the record of an imaginary representative station, called the *station year*. This method is, thus based upon the assumption, that all the stations situated in a meteorologically homogeneous area will experience similar rainfall frequencies, and that, if these stations are widely enough separated, the total of rainfall experienced at all of the stations will be similar to the experience of any one station, over a period of record equal to the sum of periods at the individual stations.

The limitations of this method are the same as that of the method $N = T \,.\, m$, plus an assumption that all the stations should lie in a meteorologically homogeneous area.

7.19. Meteorologically Homogeneous Areas

If the probability of occurrence of a storm of a given intensity is the same throughout an area, then the area is said to be meteorologically homogeneous. Strictly speaking, the rainfall, therefore, is the same at all points in such an area. Thus, if the frequencies as well as the intensities of all the rains are the same throughout, the total annual rain or the average annual rain will also be the same at all places in such an area. Hence, if there are two places which are meteorologically homogeneous, their average annual precipitations must also be the same. The converse is not true, as the two places having the same mean annual rainfalls may not be meteorologically homogeneous, because the intensities of storms constituting the total mean annual rainfall may not be the same in both the cases.

However, if the two places have the same mean annual rainfall and the same average number of days of rainfall per year, they are regarded as meteorologically homogeneous.

7.20. Intensity-Duration-Frequency Curves (IDF curves)

Rainfall of a place can be completely defined if the intensities, durations and frequencies of the various storms occurring at that place are known. Whenever, an intense rain occurs, its magnitude and duration is generally known from the meteorological readings. Thus, at a given station, the magnitudes of the isolated rains of various durations, such as 5 minutes, 10 minutes, 15 minutes, 30 minutes, 60 minutes, 120 minutes, etc., are generally known. This available data can be used to determine the frequencies of the various rains, as described earlier (by using the equation $N = T \cdot m$).

This frequency-data for storms of various durations, so obtained, can be represented by *intensity-duration frequency curves.* An intensity duration frequency curve is a plot of average rainfall intensity* in cm/h and the duration in minutes. A typical intensity-duration curve of a particular frequency is shown in Fig. 7.31. Different such curves can

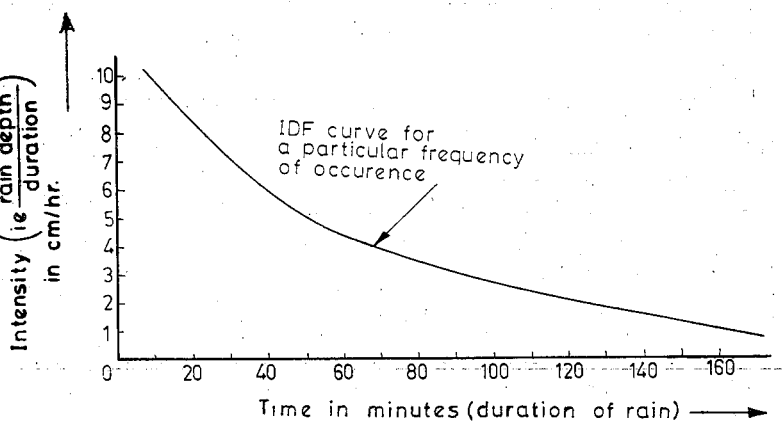


Fig. 7.31. A typical IDF curve.

be plotted for different frequencies, as shown in Fig. 7.32 and 7.33.

*rainfall depth is averaged over the duration i.e. $\frac{\text{rainfall depth}}{\text{duration}}$ = av. rainfall intensity in that duration.

Example 7.10. *The various data were obtained for rains of various durations at a station for 31 years. The records were analysed and eleven worst storms of various durations have been stipulated in their decreasing order, as shown in Table 7.14.*

Table 7.14

5 minutes		10 minutes		15 minutes		30 minutes		60 minutes		90 minutes		120 minutes	
Year	Ppt. in cm	Year	Ppt. in cm	Year	Ppt. in cm	Year	Ppt. in cm	Year	Ppt. in cm	Year	Ppt. in cm	Year	Ppt. in cm
1908	0.85	1908	1.20	1908	1.40	1908	1.74	1908	2.15	1908	2.46	1915	2.97
1921	0.76	1915	1.04	1915	1.18	1904	1.55	1904	1.92	1915	2.38	1908	2.63
1915	0.73	1921	0.93	1904	1.11	1915	1.36	1915	1.70	1904	2.14	1904	2.34
1934	0.72	1904	0.88	1921	1.03	1921	1.22	1926	1.45	1921	1.81	1921	2.12
1929	0.66	1926	0.84	1926	0.97	1926	1.18	1921	1.40	1926	1.65	1926	1.83
1926	0.62	1934	0.80	1934	0.92	1931	1.10	1914	1.33	1914	1.50	1917	1.64
1931	0.51	1929	0.78	1929	0.90	1934	1.05	1931	1.25	1931	1.40	1914	1.55
1904	0.45	1931	0.68	1931	0.82	1929	1.01	1934	1.20	1917	1.36	1931	1.51
1917	0.36	1911	0.52	1911	0.67	1911	0.95	1929	1.14	1934	1.34	1934	1.46
1914	0.28	1917	0.51	1917	0.62	1917	0.83	1911	1.11	1929	1.27	1929	1.41
1911	0.21	1914	0.39	1914	0.50	1914	0.79	1917	1.09	1911	1.23	1911	1.34

Plot the intensity-duration curves for storms of frequencies (a) 11; (b) 1.4; (c) 1.0.

Solution. We will, first of all, calculate the frequencies of the various storms by using the equation $N = T \cdot m$, as shown in Table 7.15, using total number of years $N = 11$, and $T = \frac{11}{m}$.

Table 7.15
Calculation Table for Example 7.10

S. No.	*5 minutes ppt*	*10 minutes ppt*	*15 minutes ppt*	*30 minutes ppt*	*60 minutes ppt*	*90 minutes ppt*	*120 minutes ppt*	*m = ranking of storm*	*T = frequency $= \frac{N}{m} = \frac{11}{(9)}$*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
1.	0.85	1.20	1.40	1.74	2.15	2.46	2.97	1	11
2.	0.76	1.04	1.18	1.55	1.92	2.38	2.63	2	5.5
3.	0.73	0.93	1.11	1.36	1.70	2.14	2.34	3	3.7
4.	0.72	0.88	1.03	1.22	1.45	1.81	2.12	4	2.8
5.	0.66	0.84	0.97	1.18	1.40	1.65	1.83	5	2.2
6.	0.62	0.80	0.92	1.10	1.33	1.50	1.64	6	1.8
7.	0.51	0.78	0.90	1.05	1.25	1.40	1.55	7	1.6
8.	0.45	0.68	0.82	1.01	1.20	1.36	1.51	8	1.4
9.	0.36	0.52	0.67	0.95	1.14	1.34	1.46	9	1.2
10.	0.28	0.51	0.62	0.83	1.11	1.27	1.41	10	1.1
11.	0.21	0.39	0.50	0.79	1.09	1.23	1.34	11	1.0

Average intensities for different durations such as 5, 10, 15, 30, 60, 90 and 120 minutes are then worked out for three different frequencies of 11, 1.4 and 1.0 years, as desired in the question, and shown in Table 7.16.

Table 7.16

Duration in minutes (1)	*Ppt. in cm from Table 7.15* (2)	*Av. intensity in cm/hr; Col. (2)* $\times \frac{60}{Col.\ (1)}$ (3)
Part (*a*) Frequency = 11 years		
5	0.85	12.20 (*i.e.* 0.85 × 60/5)
10	1.20	7.20 (*i.e.* 1.2 × 60/10)
15	1.40	5.60 (*i.e.* 1.4 × 60/15)
30	1.74	3.88 (*i.e.* 1.74 × 60/30)
60	2.15	2.15 (*i.e.* 2.15 × 60/60)
90	2.46	1.64 (*i.e.* 2.46 × 60/90)
120	2.97	1.42 (*i.e.* 2.97 × 60/120)
Part (*b*) Frequency = 1.4 years		
5	0.45	5.40
10	0.68	4.01
15	0.82	3.28
30	1.01	2.02
60	1.20	1.20
90	1.36	0.90
120	1.51	0.75
Part (*c*) Frequency = 1.0 year		
5	0.21	2.52
10	0.39	2.34
15	0.50	2.00
30	0.79	1.58
60	1.09	1.00
90	1.23	0.82
120	1.34	0.67

The three curves corresponding to three frequencies can then be plotted by using the above values, as shown in Fig. 7.32.

Thus, for a given storm and of a given frequency of occurrence, an intensity-duration curve can be plotted easily, as described in the above example. Highest curve in Fig. 7.32 represents intensity duration for a storm of 11 years frequency, and such a curve can be re-used to determine the average intensity of any other 11 years frequency rain for any desired duration, which when multiplied by that duration will directly indicate the rainfall depth, which could help in calculating the excess rainfall depth, and hydrograph of DRH by application on unit hydrograph, as discussed in article 7.50.4. IDF curves can, thus, prove to be an important tool in computation of runoff from a basin for a rainfall of a particular frequency and duration.

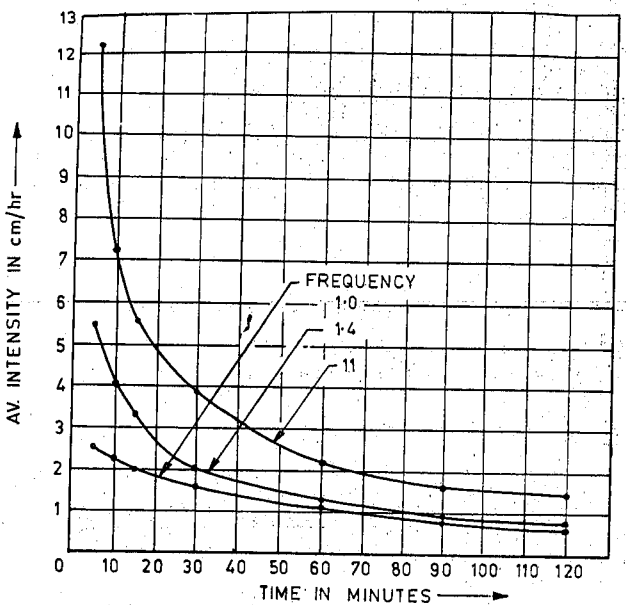


Fig. 7.32. Intensity-duration curves for different frequencies. (Example 7.10)

7.20.1. Computation of IDF curves from Long Duration Storms. When several *isolated storms* of different durations are not available on record, an IDF curve can be plotted with the data of a single long duration storm. Say for example, an IDF curve was plotted in Example 7.10 from the known rainfall depths of 7 individual storms, respectively having duration of 5 min, 10 min, 15 min, 60 min, 90 min, and 120 min. This job can also be done if the mass rainfall curve of a single storm of 120 min duration is known. In such a case, the entire duration of 120 min shall be divided into several intervals, say of 10 min each, and the rainfall depths falling in each successive interval of 10 min shall be computed. *The maximum water depth falling in any of these intervals shall be chosen as the requisite maximum depth of rainfall for 10 min duration.* When this maximum depth is divided by the above duration (10 min), the *maximum average intensity* corresponding to 10 min duration shall be obtained. The above procedure shall then be used to compute maximum depth of rain that can fall in any interval of 20 min., during the entire 120 min, duration. Proceeding in this manner, maximum average* intensities for different durations can be computed and plotted to obtain a maximum I.D. curve for the given storm, as explained in Example 7.11.

However, the frequency of this curve cannot be marked, since the frequency of this storm is not known. This can of course be found out by carrying out frequency analysis of known severer storms of the past years**. All these storms shall first be analysed in the above manner to compute maximum depths or max. average intensities (max. av. intensity = max. depth/duration) for different durations. Several values of max. depths (or max. average intensities) will thus be obtained from different storms for a particular duration. This data will be subject to frequency analysis $\left(T=\frac{N}{m}\right)$ to determine their

* averaged over the duration.

** one storm is usually chosen for each year and this gives max. depth among the several storms occurring in that year.

respective frequencies. Similar procedure can be repeated for other durations also, to ultimately compute max. depths (or max. av. intensities) for different durations at different frequencies. These values can finally be plotted to obtain **standard IDF curves** of the given basin, as shown in Fig 7.33.

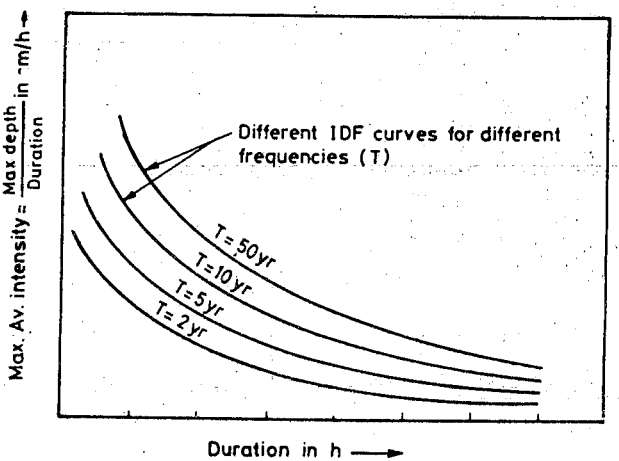


Fig. 7.33. Standard IDF curves of a basin.

Example 7.11. *The mass curve of precipitation resulted from the storm of 14th August, 1983 gave the following values :*

Hour min.	*Accumulated depth at the end of periods (in mm)*
22.00	*0.0 (Beginning of storm)*
22.05	*10.2*
22.10	*20.8*
22.15	*33.0*
22.20	*47.2*
22.25	*55.8*
22.30	*64.0*
22.35	*71.6*
22.40	*78.8*
22.45	*85.4*
22.50	*91.4 (end of storm)*

From the above storm, construct hyetograph and draw maximum intensity-duration curve. *(U.P.S.C. Engg. Services, 1984)*

Solution. To draw the requisite hyetograph, the intensities of rainfall are calculated, using uniform time interval of 5 minutes, as shown in Table 7.17. The table is otherwise self-explanatory.

Table 7.17

Time hr.min. (1)	*Cumulative rainfall in mm* (2)	*Rainfall in successive 5 min. intervals in mm* (3)	*Rainfall intensity in mm/hr* col. (3) $\times \frac{60}{5} = (3) \times 12$ (4)	*Rainfall intensity in cm/hr* (5)
22.00	0.0			
22.05	10.2	10.2	122.4	12.24
22.10	20.8	10.6	127.4	12.72
22.15	33.0	12.2	146.4	14.64
22.20	47.2	14.2	170.4	17.04
22.25	55.8	8.6	103.2	10.32
22.30	64.0	8.2	98.4	9.84
22.35	71.6	7.6	91.2	9.12
22.40	78.8	7.2	86.4	8.64
22.45	85.4	6.6	79.2	7.92
22.50	91.4	6.0	72.0	7.20

A graph is now plotted between the rainfall intensities in cm/hr obtained in col. (5) of this table and the time interval of 5 min. each, as shown in Fig. 7.34, so as to obtain the requisite hyetograph.

To draw maximum intensity duration curve, we have to calculate the maximum rainfall values for durations of 5 min, 10 min, 15 min, 20 min, ..., 50 min, as shown in Table 7.18. These maximum rainfall values are shown in dark figures in col. (3) to

Table 7.18

Time in hr-min (1)	*Accumulated rainfall in mm* (2)	*Rainfall in any possible time interval equal to*									
		5 min (3)	*10 min* (4)	*15 min* (5)	*20 min* (6)	*25 min* (7)	*30 min* (8)	*35 min* (9)	*40 min* (10)	*45 min* (11)	*50 min* (12)
22.00	0.0										
22.05	10.2	10.2									
22.10	20.8	10.6	→20.8								
22.15	33.0	12.2	→22.8	33.0							
22.20	47.2	**14.2**	→**26.4**	**37.0**	**47.2**						
22.25	55.8	8.6	22.8	35.0	45.6	**55.8**					
22.30	64.0	8.2	16.8	31.0	43.2	53.8	**64.0**				
22.35	71.6	7.6	15.8	24.4	38.6	50.8	61.4	**71.6**			
22.40	78.8	7.2	14.8	23.0	31.2	45.8	58.0	68.6	**78.8**		
22.45	85.4	6.6	13.8	21.4	29.6	38.2	52.4	64.6	75.2	**85.4**	
22.50	91.4 (end of storm)	6.0	12.6	19.8	27.4	35.6	44.2	58.4	70.6	81.2	**91.4**

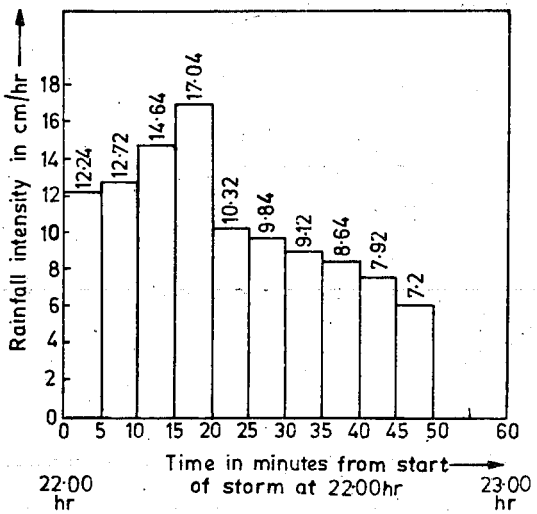


Fig. 7.34. Rain Hyetograph for Example 7.11

col. (12) of this Table. These maximum values are then used to calculate their respective maximum intensities, as shown in Table 7.19.

Table 7.19

Duration in min	*Max. rainfall in mm, from Table 7.18*	*Max. rainfall intensities*	
		in mm/hr $\text{col. (2)} \times \frac{60}{\text{Col. (1)}}$	*in cm/hr*
(1)	*(2)*	*(3)*	*(4)*
05	14.2	170.4	17.04
10	26.4	158.4	15.84
15	37.0	148.0	14.80
20	47.2	141.6	14.16
25	55.8	133.9	13.39
30	64.0	128.0	12.80
35	71.6	122.7	12.27
40	78.8	118.2	11.82
45	85.4	113.9	11.39
50	91.4	109.7	10.97

A curve is now plotted between values of col. (1) and (4) of the above table, so as to obtain the max. I.D. curve, as shown in Fig. 7.35.

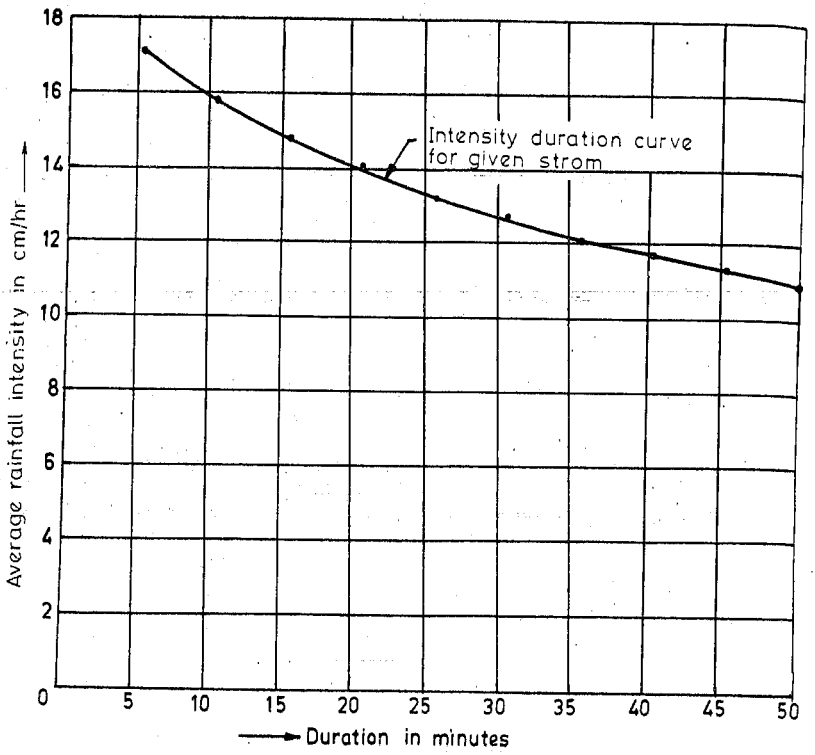


Fig. 7.35. Max. ID curve for Example 7.11.

7.20.2. Equation of an IDF curve. An IDF curve can be represented by an equation of the type :

$$p = \frac{A \cdot T^x}{(t_r + B)^n} \qquad ...(7.19)$$

where p = intensity of rain in cm/h

t_r = duration of rain in h

T = Return period or Frequency of rain in years.

A, B, n and x are all constants for a given catchment, and their typical values for a few places in India are shown in table 7.20

Table 7.20. Typical Values of Constants in eqn. (7.19)*

S. No.	*Place in India*	*Values of constants*			
		A	*B*	*n*	*x*
1	Bellary	6.16	0.50	0.694	0.972
2	Bhopal	6.93	0.50	0.189	0.878
3	Chandigarh	5.82	0.40	0.160	0.750
4	Nagpur	11.45	1.25	0.156	1.032
5	Raipur	4.68	0.15	0.139	0.928

* Based on the studies conducted by the Central Soil and Water Conservation Research and Training Institute, Dehradun.

Isopluvial maps. Instead of providing the standard IDF curves, indicating extreme rainfall intensities of different durations for different frequencies (T), IMD has standardised *isopluvial maps*- indicating extreme point rainfall values of different durations and for different frequencies over the whole country. On an Indian geographical map, therefore, *lines of equal rainfall* (called *isopluvial lines*) have been drawn, each for a particular duration and a particular frequency. Hence, different isopluvial charts indicating extreme rainfall depths for different durations of 15 min, 30 min, 45 min, 1 hour, 3 hour, 6 hour, 9 hour, 12 hour, 15 hour, etc. each for 2, 5, 10, 25 and 50 years of return period are available with IMD as well as CWC, and can be used for specific needs. 1 hour isopluvial charts of 2, 5, and 50 years return period are also enclosed in this book as plates 7.1, 7.2 and 7.3, respectively.

7.21. Probable Maximum Precipitation (PMP) Curves

The probable maximum precipitaion (PMP) may be defined as the *maximum* depth of precipitation for a given duration that may *possibly occur* on a given catchment at any time of the year. Hence, such a precipitation would result from the possible severest storm, that may result from the worst possible combinations of hydrological conditions in the area. Such a worst possible storm is called **probable maximum storm** (PMS), and will be used for designing large hydraulic structures, as discussed in article 7.51.

As is evident from its very definition, the PMP can be estimated by maximising the different parameters, such as *wind velocity, humidity*, etc. of an observed severe storm, over the given basin. Occasionally, when enough storm data for a given basin is not available, PMP can be estimated by adopting a severe storm over a neighbouring catchment, and transposing it to the catchment under consideration. Detailed discussions of such exercises is beyond the scope of this book, as the same falls in the domain of Hydro meteorology, and any standard book on that subject may by referred to, if needed.

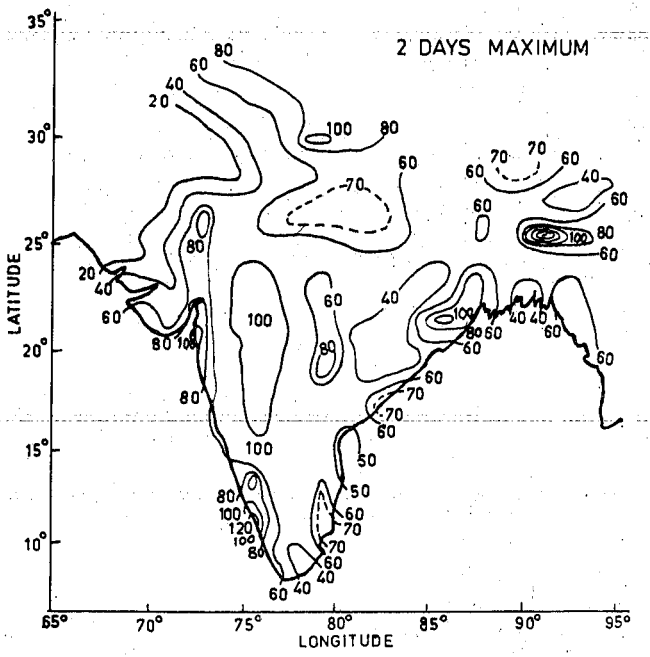


Fig. 7.36. (*a*) Heaviest rainfall in cm – 2 days duration.

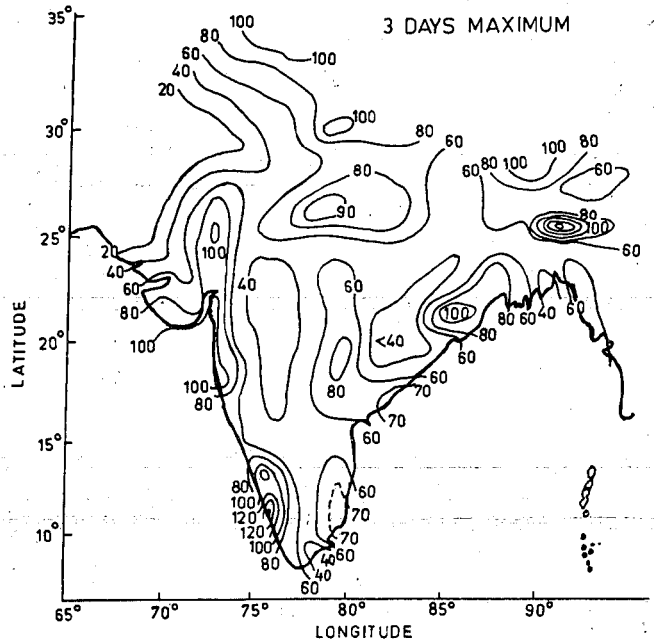


Fig. 7.36. (*b*) Heaviest rainfall in cm – 3 days duration.

Indian Meteorological Department (IMD) has prepared guiding curves, showing isohyetal lines for maximum storm depths (absolute for any frequency), that may result from the severest storm of the given duration. Fig. 7.36 (*a*) and (*b*) shows such charts for 2 days and 3 days maximum rainfall, respectively. A possible value of maximum rainfall at any given city, having 2 days or 3 days duration, can, thus be readout from these charts, easily.

SNOWFALL AND SNOW MELT

7.22. Snowfall and its Measurement

Snow is the second important part of precipitation, after rainfall. It usually occurs at high altitudes, and particularly during the winter season. It may or may not be accompanied by rainfall, depending upon the coldness of the area. In India, heavy snowfalls occur in the upper reaches of the *Himalayan rivers*, such as Ganga, Yamuna, Brahmaputra, etc., during December, January and February, as *winter precipitation.*

Snowfalls are generally responsible for giving continuous delayed supplies in the rivers, rather than giving immediate flood flows, as produced by intense rain storms. As a matter of fact, snow generally goes on falling and accumulating over the river valley and the basin, during the winter months, without giving any immediate streamflow; but later, with the onset of spring season in late March or April, when the temperature of the region starts increasing, the snow starts melting, leading to runoff in the river below. Depending upon the available snow depth, runoff continues during the summer season, and possibly till the arrival of the next winter, when snow again starts falling and accumulating. The process proves a great boon, as it ensures perennial river supplies, so very necessary for us.

Snow usually falls in the form of very light *snow flakes*, having density as low as 0.004. When such snow flakes fall and get deposited over the ground, one over the other, they get compressed, leading to increase in its density. As the depth of accumulated snow increases with the passage of time, it gets consolidated more and more, with increase in its density. Freshly fallen snow (last 24 hours), therefore usually has a density of 0.1, while the heavy snow accumulations may have as high a density as 0.4 to 0.6. When the accumulated snow depth in the river valley becomes quite high, say more than 60 to 75 m or so, it begins to move down the valley, like a flow in the stream. This snow-body is called a **glacier**, and may have as high a density as 0.9. Such a glacier may, sometimes, cause severe floods in the river below, if by chance, it happens to burst. As for example, in August 1959, a glacier burst in the remote mountain valley in the *Karokorum* mountain ranges (K_2) in Kashmir, causing a flood rise of 30 m at a distance of more than 40 km from the point of outburst. Such sudden **glacier floods**, locally called *Jokulhlaup*, may cause heavy loss of life and other valuables, and needs to be predicted well in advance. The discharge hydrograph from such a glacier flood resembles a normal storm unit hydrograph, except that the flood rise here is rather slow, and the fall is abrupt.

Since the snowfall plays an important role in supplying continuous flow in a river, and also chance floods, it is necessary to measure the snowfall, and, thus, to estimate its accumulated depths and runoff potentials.

Snowfall measurements can be divided into two categories, viz.

(*i*) *measurement of snowfall*, caused by each individual snow storm or daily snowfall; and

(*ii*) *snow surveys* to measure the accumulated snow depth deposited during the entire winter season, or at smaller intervals and to determine its density ; and hence to compute the water equivalent of the accumulated snow for estimating runoff.

Both the above types of measurements are discussed below :

7.22.1. Snow Gauging. The *snow gauging* can be carried out in the following two ways :

(*i*) by measuring the depth of snow, actually collected on an open *level platform* ; and

(*ii*) by measuring the depth of snow, caught in a *shielded gauge vessel*, after levelling the accumulated snow without compressing.

The above snow gauging is recorded every day, during the entire snowfall season, usually at 8.30 A.M., as is done at a rain gauge station.

(*i*) The **open platform method** is more suitable in the absence of strong winds, and in areas where snowfalls occur unaccompanied by rainfalls. The method consists of constructing a permanent level pucca platform, 2 m in square, at a suitably located gauge site. The top of this cement concrete platform (75 mm thick C.C.) is kept at the ground level, and a graduated wooden square rod, 50 mm in section, 4 m long is usually installed at the centre of this platform. Lower 1 m length of this wooden rod is embedded into the ground and the pucca platform ; while, the upper 3 m length of the rod is exposed and graduated on all the four sides, for taking *stake* readings. Evidently, the reading at

the top level of the platform will be zero, and increasing upwards up to 3.00 m on the wooden stake, called **snow stake**, which may also be supported on sides by angle iron struts. This *wooden gauge* would be something like your levelling staff.

Besides the above permanent gauge, *moveable hand gauges* are also kept at the gauging station. Such a hand gauge, called a **hand stake**, is nothing but a graduated wooden pole, usually 25 mm square in section, with tapering bottom, 3 m long and provided with a metallic shoe at its bottom, as shown in Fig. 7.37. Such hand gauges are used by the observer for measuring the depth of accumulated snow at the four corners of the platform, in addition to taking the reading on the central fixed gauge. The arithmetic mean of all the five readings shall be recorded as the snowfall of the previous day. This snowfall depth is usually divided by 10, so as to obtain the depth of *equivalent rainfall* or *water equivalent of the snowfall*, on the assumption that the density of snowfall collection, if measured over 24 hours, shall be 0.1.

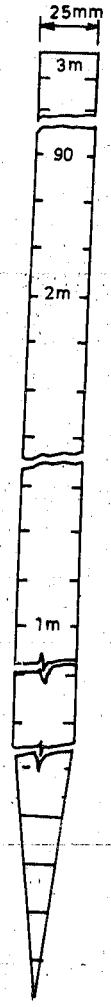


Fig. 7.37. Hand stake.

This method of measuring snowfall also requires ensuring that *no old snow is measured the next day*. This can be done by laying suitable covers of suitable material on top of the snow surface, each day, and measuring the depth down up to these covers, the next day.

In areas of strong winds, such unshielded snow collection measurements may not give precise results, since snowfall may drift either away from the platform or more towards the platform. Hence, for such windy areas, we may either have to install a large number of such gauging platforms, so as to get a better mean, or may have to use shielded snow gauges.

Moreover, in remote and inaccessible areas, special types of snow stakes, which can be read from a low flying aircraft, may be used.

(*ii*) A **shielded snow gauge** is a standard rain gauge, protected and provided at top, with a specially designed sheet metal shield, as shown in Fig. 7.38, which shows a *modified Nipher shield.*

Snow is, sometimes, measured in an ordinary *weighing type of a rain gauge* (Fig.7.13), properly equipped with a shield to reduce the effect of wind.

The USBR has also developed a snow gauge with a heated intake, which prevents *bridging of snow* over the gauge intake.

As a matter of fact, the direct measurement of snow depth at a single station is generally not useful, because of likely inaccurate estimates of the distribution over large

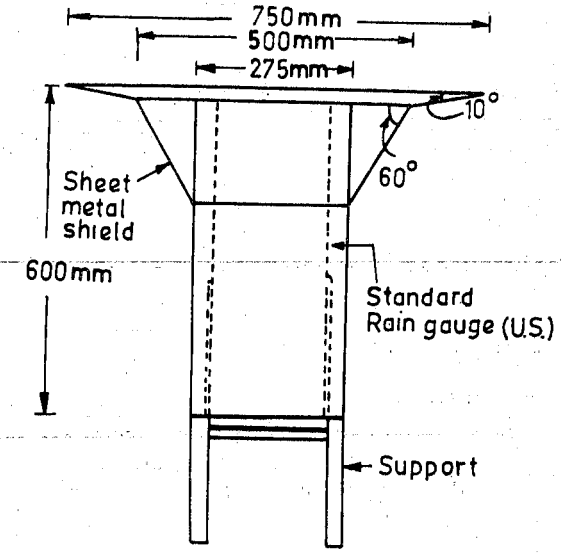


Fig. 7.38. Shielded snow-gauge (modified Nipher shield).

areas, as the measured depth may be highly unrepresentative due to drifting or blowing. To overcome this problem, snow surveys are undertaken, as discussed below:

7.22.2. Snow Surveys. Snow surveys consist of collecting *full depth snow samples*, which prove to be quite helpful in determining the snow depths available, and also the density of such accumulations, thus to estimate the amount of water expected due to melting of snow during the year. Such surveys are, therefore, generally conducted either at suitable intervals, such as weekly, fortnightly or monthly; or *annual surveys* are conducted at the start of the spring season (*i.e.* in March or April in India), to know the availability of water during the coming summer season.

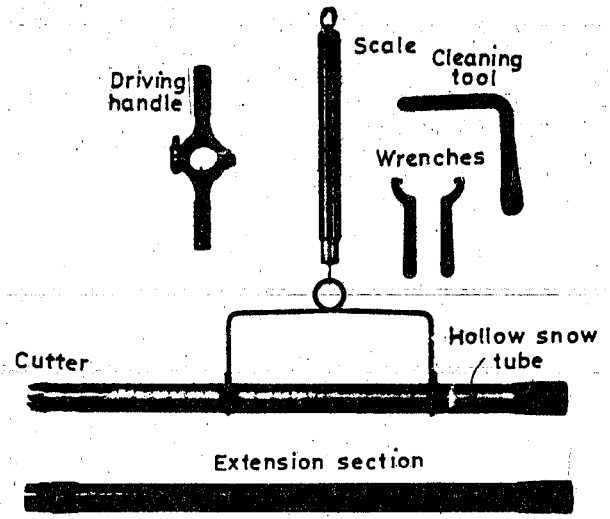


Fig. 7.39. Snow tube.

Such surveys are conducted along predecided representative routes, called *snow courses*; and snow samples are collected along such routes at intervals of say 15 to 20 m. The hollow *sampling tube* called **snow tube** of diameter 40 to 90 mm, provided with a cutting edge (Fig 7.39) is driven vertically into the snow pack by rotating it, till it reaches the bottom of the snow pack. Extension tubes are added, if required. The depth of snow is noted from the graduations on the outer surface of the tube. The tube along with embedded snow in the tube, is then withdrawn, and weighed. The weight of snow trapped in the tube, divided by the unit weight of water, will evidently represent the water depth available. The density of the snow sample (ρ_s) can also be calculated by dividing the mass of the collected snow by the volume of snow (where volume = depth × area of cylindrical hollow of the sampling tube). The snow depth (D_s) multiplied by ρ_s will represent the equivalent water depth.

Special types of **radioactive snow gauges** are also available, which can be used to determine the variation of snow density with depth, within the snowpack, thus determining the available snow depths and their water-equivalents.

Radar and satellite observations do help in estimating the basin area covered with snow, and thus, to estimate the volume of flow expected from the area.

7.22.3. Empirical Formulae for Determining Snowmelt. The following empirical relations have been developed for estimating the daily streamflow likely to be caused by snow accumulations :

(1) **U.S. Army Corps of Engineers Equations**

(*a*) *For open sites*,

$$M_{24} = 0.03\ (9\ T_m + 40)\ ;\ \text{and} \qquad \ldots(7.20)$$

$$M_{24} = 0.02\ (T_{max} + 25) \qquad \ldots(7.21)$$

where M_{24} = daily (24 h) spring time snowmelt in cm/day

T_m = Mean daily temperature in °C

T_{max} = Maximum daily temperature in °C.

(*b*) *For forested sites*

$$M_{24} = 0.025\ (9\ T_m)\ ;\ \text{and} \qquad \ldots(7.22)$$

$$M_{24} = 0.02\ (9\ T_{max} - 50) \qquad \ldots(7.23)$$

(2) **Light's Equation.** It states that

$$M = \frac{\rho\ K_0^2}{80 \log_e\left(\frac{a}{z_0}\right) \log_e\left(\frac{b}{z_0}\right)} \left[V\ (C_p \cdot T + e - 6.11)\ \frac{423}{p} \right] \qquad \ldots(7.24)$$

where M = snowmelt in cm/sec

K_0 = Von Karman's coefficient = 0.38 to 0.40

ρ = density of air

z_0 = roughness parameter = 0.25

v = wind velocity at anemometer level

a = level at which wind velocity is measured in millibar (mb)

e = vapour pressure of air in mb
T = Temp. of air °C
b = level at which temperature and vapour pressure of air are measured, which must be the same and taken at the same time
p = atmospheric pressure in mb
C_p = Sp.heat of air at constant pressure = 0.24

The above equation gives the theoretical value of snow melt at a point, and hence, in order to obtain the snowmelt from a drainage basin, the surface roughness and forest cover of the drainage basin should be given due consideration. These two factors can be included in a basin constant K, and, hence, the

$$Basin\ snowmelt = K.M. \qquad ...(7.25)$$

Moreover, the above equations should be used over a maximum of 6 hr. time interval, because the use of average values of air temperature and dew point for periods longer than 6 hour, may yield different results from those obtained by summing up the computed values for 6 hour periods.

The above Light's equation is, infact, based on computing the snowmelt, caused by different **heat sources**, such as *convection, condensation, radiation*, and *warm rain*. The snowmelts, caused by each of these four sources of heat, can be separately estimated, over each 6 hr period, and then added to get the total flow during the said 6 hr.

The *individual snowmelt equations* are given below:

(*i*) *Snowmelt by convection*

$$M_6 = KVT \qquad ...(7.26)$$

where M_6 = depth of water from snowmelt in 6 hr period, due to heat of convection
V = wind velocity in km/h
T = air temperature in °C
K = a constant, involving the latent heat of ice, exposure of instruments, air density, conversion of units, and certain considerations involved in the theory of turbulence. Its usual value varies between 0.003 to 0.006.

(*ii*) *Snowmelt by condensation*

$$M_6 = KV(e - 6.11) \qquad ...(7.27)$$

where M_6 = depth of water from snowmelt in 6 hr period, caused by heat of condensation
V = wind velocity in km/h
e = Vapour pressure of air in millibar (*m*b)
6.11 = numerical value, which is the saturation vapour pressure in mb over ice at 0°C
K = an empirical constant, which equals about 0.005 corresponding to data of temperature at 1.25 m and wind at 4.5 m height, respectively.

(*iii*) *Snowmelt by radiation is given by Wilson's equation*, as:

$$M_{12} = M_0 (1 - 0.75\, m) \qquad ...(7.28)$$

where M_{12} = Snow melt in 12 hr on a cloudy day (noon to mid night or mid night to noon)

M_0 = Snowmelt occurring in half day in clear weather; standard values given in table 7.21

m = degree of cloudiness, which is 0 for clear whether, and 1.0 for complete overcast.

Table 7.21. Suggested Monthly values of 12-h snowmelt in clear weather (M_0) within latitude 40 to 48° (after Wilson)

Month	M_0 *(cm)*
March	0.89
April	1.07
May	1.22
June	1.35

(*iv*) *Snowmelt by rainfall* falling over the snow:

$$M_1 = \frac{P \cdot T_r}{80} \qquad ...(7.29)$$

where M_1 = snowmelt in cm/h

P = rainfall in cm

T_r = Temp. of rain water in °C, assumed as equal to the temperature of wet bulb.

Example 7.12. *During a cloudy 12-h period on 12.4.90 at a place located on altitude of 48° N, and a height of 4000 m above sea level, where an average ripe snowpack is existing; the following weather averages were recorded*

Air temperature = *12 °C*
Mean wind velocity = *16 km/h*
Relative humidity (h) = *60%*

Estimate the snowmelt caused by different heat sources, and the total snowmelt during 12 h period, (a) if only negligible rainfall had occurred during the said period. (b) If the area of the snowpack has been estimated by satellite imageries to be 12 km^2, then estimate the expected average streamflow due to melting of snow.

Solution.

(*i*) *Snowmelt depth caused by convection:* It is given by equation (7.26) as :

$$M_6 = KVT$$

where K = basin constant= 0.005 assumed between 0.003–0.006

V = wind velocity in km/h = 16 km/h (given)

T = air temp, in °C = 12°C(given)

$\therefore \quad M_6 = 0.005 \times 16 \times 12 = 0.96$ cm/6 h

$\therefore \quad M_{12} = 2M_6 = \mathbf{1.92\ cm/12\ h.}$ **Ans.** ...(*i*)

(*ii*) *Snowmelt depth caused by condensation.* It is given by equation (7.27) as:

$$M_6 = KV(E - 6.11)$$

where $K = 0.005$ (already assumed for the given basin)

$V = 16$ km/h (given)

e = vapour pressure of air in *mb*, and given by R.H.; as

$$R.H. = \frac{\text{V. Pressure}}{\text{Saturation V. Pressure}}$$

or $$60\% = \frac{e}{e_s}$$

where e_s at 12°C from Table 7.23 is

$= 10.54$ mm of Hg $= 10.54 \times 1.33$ mb $= 14.0$ mb

or $$0.6 = \frac{e}{14}$$

$$e = 0.6 \times 14 = 8.4 \text{ mb}$$

$\therefore \quad M_6 = 0.005 \times 16\,(8.4 - 6.11) = 0.18$ cm/h

Snowmelt for 12 h $= M_{12} = 2M_6 = 2 \times 0.18 = \mathbf{0.36\ cm/12\ h.}$ **Ans.** ...(*ii*)

(*iii*) Snowmelt caused by radiation. It is given by eqn. (7.28) as:

$$M_{12} = M_0\,(1 - 0.75\,m)$$

where $M_0 = 1.07$ assumed from table 7.21

$m = 1$ for fully cloudy day

$\therefore \quad M_{12} = 1.07\,(1 - 0.75) = \mathbf{0.27\ cm/12\ h}$ **Ans.** ...(*iii*)

Total snowmelt in 12 h

$= (i) + (ii) + (iii) = 1.92 + 0.36 + 0.27 = \mathbf{2.55\ cm/12\ h.}$ **Ans.**

(*b*) Average snowmelt as discharge

= Snowmelt as depth × Area of snowpack

$$= \frac{2.55}{100} \times \frac{(12 \times 10^6)}{12 \times 3600} = \mathbf{7.08\ m^3/sec.}$$ **Ans.**

HYDROGRAPH OF RUN-OFF

7.23. Rainfall-Runoff Process

When a rain starts falling, it is first of all intercepted by buildings, trees, and other objects, which prevent it from reaching the ground. This quantity is known as *rainfall interception*. Since this quantity is generally very small, it is not of much importance for intense rains ; but many a times, large potions of lighter rains are disposed of in this manner. The difference between the total rainfall and that which is intercepted is called *ground rainfall.*

Secondly, when the rainfall rate exceeds the interception rate, water starts reaching the ground and *infiltration* into the sub soil starts. The *maximum rate* at which the soil in a given condition can absorb water is known as its infiltration capacity (*f*)*.

Thirdly, the excess rainwater gets collected into the innumerable small and large depressions existing in the basin, filling them to their overflow levels. This quantity is known as *depression storage.* All this storage is either evaporated or used by vegetation, or later infiltrates into the soil. None of it appears as *surface runoff.*

If after the depression storage is filled, the rain intensity (*p*) continues to exceed the infiltration capacity of the soil (*f*), the difference appears as *rainfall excess*, which initially accumulates on the ground as *surface detention* (*D*), and then flows as *overland flow* on the basin surface before entering a stream channel. The water that reaches the stream channel of a basin in this manner is called *surface runoff* (*SRO*) or *direct runoff (DRO). The surface runoff can, therefore, occur only from those storms, which can contribute to excess rainfall, and are simply not dissipated in fulfilling the interception, depression storage, and infiltration needs of the basin.*

Excess rainfall, also called effective rainfall, can thus be represented as :

$$\text{Excess rainfall} = \text{Rainfall} - \text{Interception} - \text{Depression storage} - \text{Infiltration} \quad ...(7.30)$$

The sum total of interception and depression storage for a basin is usually called *the initial loss*, or the *basin loss*, or the *initial basin recharge.* Excess rainfall can therefore be represented as :

$$\text{Excess rainfall} = \text{Rainfall} - \text{Initial Basin loss} - \text{Infiltration} \quad ...(7.31)$$

The sum total of initial basin loss and infiltration, is called *potential infiltration.* Since for intense rains, the initial loss is very small as compared to excess rain, it is usually ignored in hydrological analysis of such rainfalls; or is considered to be included in the infiltration itself. The excess rainfall is, thus, represented as :

$$\text{Excess rainfall} = (\text{Rainfall} - \text{Infiltration}) \text{ ; where infiltration includes initial loss} \quad ...(7.32)$$

The excess water flowing as overland flow or channel flow in a watershed, finally appears as surface runoff at the outlet of the basin, but only after filling the capacity of its stream channel. This storage at any instant is known as *channel storage*, and is likely to reduce the peak rate of surface runoff at the basin outlet.

Initial and Residual rain. The rain that falls in the beginning of a storm before the depression storage is completely filled is called the *initial rain,* and the rain that falls near the end of the storm at a rate less than the infiltration capacity is called the *residual rain.* The intervening period is the *net supply interval.* The infiltration occurring after the net supply interval is called the *residual infiltration.* It consists of residual rainfall plus that portion of surface detention which is on the ground at the end of net supply interval but later infiltrates. The total rainfall excess is thus equal to surface runoff plus the difference between the residual infiltration and residual rainfall. However, because

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* The rain will enter the soil at full capacity rate only when the rainfall rate exceeds the infiltration capacity ; and when the rainfall rate is lesser, then the actual *infiltration rate* will be lesser than *f*, and equal to the rainfall rate minus interception rate.

the latter quantity is usually small, *the excess rainfall is considered to be equal to the surface runoff.*

7.24. Runoff and Surface Runoff

Runoff and surface runoff are two different terms and should not be confused. Runoff (RO) includes all the water flowing in the stream channel at any given section. While the surface runoff (SRO) includes only the water that reaches the stream channel without first percolating down to the water-table. *Surface Runoff* is also called as the **Direct Runoff (DRO)**; and *Runoff* is also called the *Discharge* or the **Streamflow.**

7.25. Yield of a Drainage Basin

Yield of a drainage basin is the same as runoff, with the only difference that it is expressed over long periods, while the Runoff is expressed for short periods. Thus, the Yield is generally expressed as the total volume of water flowing per year, such as $M.m^3$/year; while the Runoff is the instantaneous rate, and is generally expressed as volume of water flowing at a given section of the river in one second or one hour, such as in m^3/s or m^3/h.

7.26. Constituents of Surface Runoff

Surface runoff (SRO) as defined earlier, is the water that reaches the stream channel without first percolating down to the watertable. This surface runoff, may for certain areas, be divided into two parts :

(1) Water that flows directly over the ground surface, called *True surface runoff.*

(2) A part of the water that infiltrates through the soil, moves laterally, and before joining the watertable, joins the river channel, as shown in Fig. 7.40. This quantity is known as *Sub-surface runoff* or *Sub-surface storm flow*. It behaves nearly like the surface runoff and not like the ground water flow, because it reaches the stream so quickly that it is difficult to distinguish it from true surface runoff. On the other hand, the ground water flow is often times long delayed before it reaches the stream.

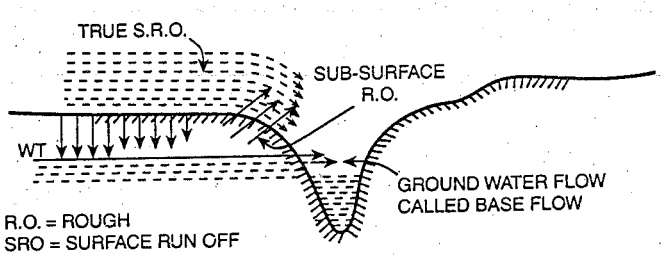


Fig. 7.40. Constituents of Surface runoff and of runof.

For this reason, sub-surface runoff is always treated as a part of surface runoff.

7.27. Constituents of Runoff

The runoff of a river stream consists of the following three constituents:

1. *Direct precipitation over the surface of the stream.*
2. *Surface runoff* consisting of true surface runoff and sub- surface stormflow.
3. *Ground water inflow*, popularly called the **base flow.**

The first part provides a very small portion of the total flow. This amount depends upon the lake area of the stream channel. Even for rivers having exceptionally large lake areas, the evaporation from those water surfaces may nearly or more than balance the precipitation on them. This factor can, therefore, be ignored. Hence, the runoff of a river can be supposed to be consisting of surface runoff and ground water inflow only. Hence, we can write

$$\text{Runoff} = \text{Surface runoff} + \text{Ground water inflow } (i.e. \text{ Base flow}). \quad ...(7.33)$$

7.28. Hydrograph of Stream-flow

A *hydrograph* or a *Runoff hydrograph* is the graphical presentation of the discharge flowing in a river at the given location, with the passage of time. It is, thus, a plot between time (on X-axis), and discharge (on Y-axis), such as shown in Fig. 7.41.

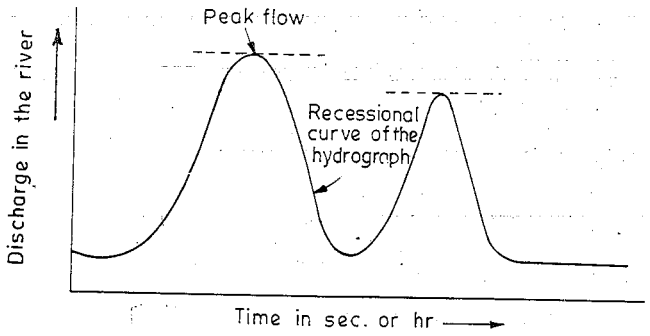


Fig. 7.41. A typical hydrograph of runoff.

Such a graph is capable of representing discharge fluctuations in the river at a given site over a given time period, and can also indicate the *peak flow*, which will govern the design of the given hydraulic structure at that site.

The maximum flow in the river due to any given storm is known as the peak flow. This peak flow will be different for different kinds of storms.

7.29. Base Flow

Runoff hydrograph for a river, as stated above, is the graphical presentation of the river discharge with time. The river discharge constitutes of surface runoff and ground water inflow (called base flow).

In order to obtain the surface runoff graph, we must subtract the base flow from the runoff graph. It, therefore, becomes necessary to understand the movements of ground water flow into a river.

As a matter of fact, the ground water may sometimes enter a river, and sometimes get out of the river. Depending upon whether the water is **entering** the channel or *going out of the channel*, the channel may be called as the **effluent stream** or the *Influent stream*, respectively. The same channel may behave as an effluent stream as well as an influent stream with the changing seasons.

The reasons responsible for these two kinds of movements are explained below :

Consider a river cross-section as shown in Fig. 7.42. Let *lm* be the position of watertable in the area adjoining the river, and let *ab* be the water level in the river. Let the water level *ab* be lower than the watertable *lm*. Such a condition may occur in the summer season, after the rains have passed away.

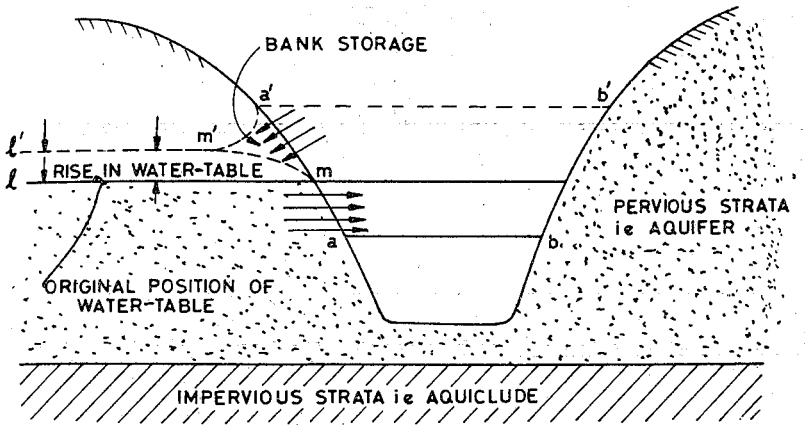


Fig. 7.42

Under this condition, since the water level in the river is lower than the watertable, a flow will start from beneath the watertable into the stream due to the difference of hydrostatic pressure between the banks and the stream. This ground-water *inflow* will continue till the two water levels are equalised. Many of the rivers get their supplies during the summer season through this process of ground-water inflow.

During the rainy season, when heavy rains exceeding the soil moisture defficiency, fall on the surface, causing sufficient amount of surface run-off, the existing conditions can be visualised as follows :

The watertable as well as the river-water surface will go up. But the rise in the river-water will be much more than the corresponding rise of the watertable. Let *l'm'* be the new position of the watertable, and let *a'b'* be the new water surface in the river. As soon as such a condition occurs, there will exist a greater hydrostatic pressure in the stream than at the banks, and hence, outflow will start from the stream. In other words, the ground-water inflow stops, and the direction of flow reverses, creating a *Bank storage* represented by *a'm'm* (Fig. 7.42). This *outflow* will continue till the water level in the river channel is higher than the ground watertable. As soon as the water level in the river channel falls below the ground watertable, the direction of flow again reverses, and the ground water inflow into the stream channel restarts. And for a time, because of the bank storage, this inflow of water is considerably increased. As soon as the bank storage is drained out, the water will flow in a normal manner.

Hence, it can be concluded that during the peak stage of a river, there exists an outflow from the river into the ground-water reservoir, but as soon as the peak starts falling, the flow reverses and the inflow of the ground water into the river starts taking place. Graphically, it can be represented as shown by dotted line in Fig. 7.43. The portion of the curve below the horizontal axis represents the *outflow* from the river flow, and the portion above it represents the *inflow*. Since the outflow from the river occurs for very short time periods and acts like infiltration, it is generally ignored in all the hydrological computations, while the inflow into the river occurring over long periods

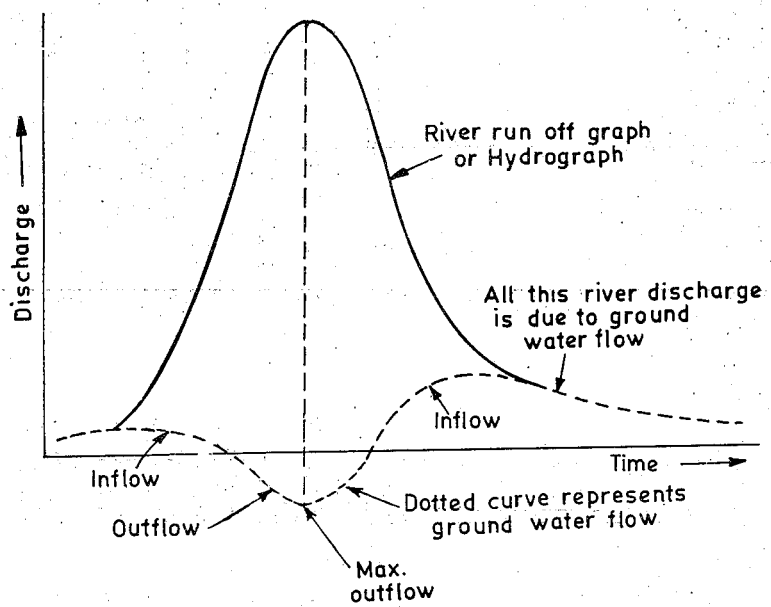


Fig. 7.43. Effect of ground water flow on river discharge.

of time and responsible for the increased runoff in rivers, is duly accounted for. This *inflow* is therefore generally referred to as the **base flow.**

7.30. Ground Water Depletion Curve or Base Flow Curve

When there is no surface runoff either from the rains or from the melting snow, the river water is derived from the ground water. This results in a steady lowering of the watertable and a constantly reducing streamflow (discharge) until a rain occurs to produce ground water accretion or surface runoff. If after a heavy rain (when watertable is at a maximum height), no further rain occurs, streamflow may be supplied entirely by the base flow, till watertable falls below the lowest bed level of the stream channel, and in such a condition, there will be no flow in the river. The hydrograph of the river during this period (*i.e.* from the time when watertable is at maximum height to the time when there is no flow) is nothing but the hydrograph of ground water inflow in the river, and is known as the *Ground water depletion curve* or the *Base flow curve.* A typical ground water depletion curve is shown in Fig. 7.44.

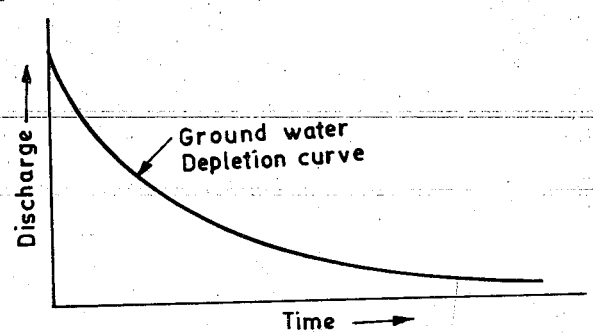


Fig. 7.44. Typical ground water depletion curve.

As explained above, the hydrograph of a river is the representation of runoff of a river. The runoff is the sum of surface runoff and ground water flow. When S.R.O. is zero, the runoff is equal to the ground water flow. Therefore, during the periods of zero S.R.O., the hydrograph of a river will represent the hydrograph of the ground water flow into the river, *i.e.* ground water depletion curve.

Hence, the ground water depletion curve must be represented by some portion of the annual hydrograph of the river. In fact, the recession curve of the annual hydrograph represents nothing but a part of the ground water depletion curve.

Only a very long rainless period ensures the possibility of complete development of ground water depletion curve. But in actual practice, in a region of moderate or high precipitation, there never occurs a rainless period of such a long duration, as to cause the complete development of ground water depletion curve. A striking example of ground water depletion curve is the hydrograph of *Lualaba River* in Belgian Congo. This is a tropical river having marked seasonal rainfall, and no rainfall during May to September. During this period of May to September, the total runoff is based upon the ground water inflow. A typical annual hydrograph of this river is of the type, shown in Fig. 7.45. The portion *CD* of its hydrograph represents the ground water depletion curve or *base flow curve.*

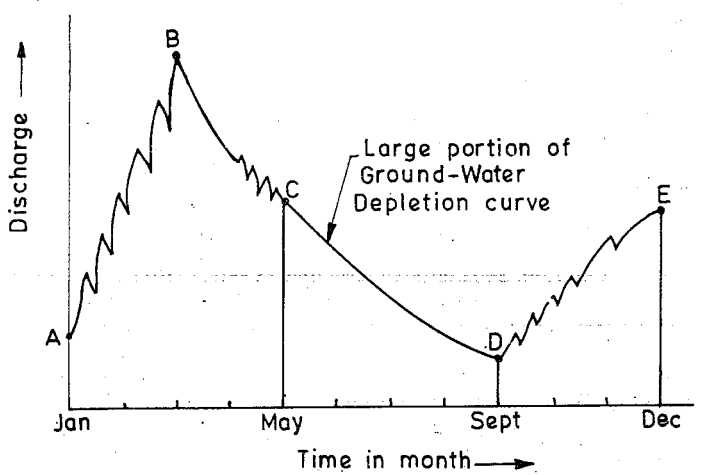


Fig. 7.45. Annual hydrograph of Lualaba river.

Similarly, majority of rivers in Indian Peninsula run on ground water supply after monsoons during Oct to May. This off-season flow is called here as *post monsoon flow.*

7.31. Separation of Base Flow from the Hydrograph of River to obtain Direct Runoff Hydrograph

If the water flowing into a river from the ground water is deducted from the actual river discharge, we can get the graph of the river surface runoff, provided the direct precipitation over the river is ignored.

Therefore, in order to separate the surface runoff, the ground water inflow curve should be superimposed over the runoff hydrograph. The figure enclosed between these two curves will then represent the S.R.O. hydrograph or the direct runoff hydrograph, popularly written as DRH.

But the evaluation of the ground inflow curve is not easy, and hence to separate the base inflow, a straight line such as *ab* is generally drawn on the hydrograph of river runoff (Fig. 7.46).

The point *b*, however, cannot be precisely determined, but this is not of vital importance, so long as the same procedure is adopted throughout. This point may be taken as the point of greatest curvature* near the lower end of the recession side of the hydrograph. Truely speaking, the point *b* represents the point where surface runoff ceases, and, therefore, after this point *b*, the runoff hydrograph will be nothing but the ground water depletion curve. And hence the portion *bc* infact represents the ground water depletion curve.

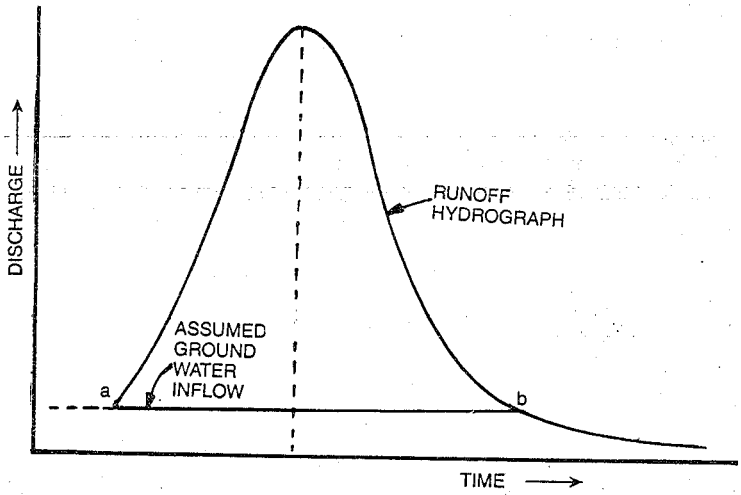


Fig. 7.46. Separation of base flow to obtain DRH.

Therefore, if sufficient records are available, and if from these records the ground water depletion curve is plotted on a transparent paper, which is moved on the runoff hydrograph till this curve coincides with portion *bc* of the hydrograph, we can easily fix the point *b*. In this method of fixing the point *b*, care should be taken to see that the ground water depletion curve has been plotted on the same scale as that of the river hydrograph.

7.32. Annual Hydrographs of Perrenial, Intermittent and Ephemeral Streams.

A study of the annual hydrographs of streams can help us to classify the streams into the following three categories :

(i) Perennial streams;

(ii) Intermittent streams; and

(iii) Ephemeral streams.

Perennial streams carry some flow throughout the year (Fig. 7.47), and hence usually carry a considerable amount of *snow melt* from upper hilly reaches, or the *base flow* due to higher position of the ground watertable than the water level in the river. Ganga & Brahamaputra rivers are the excellent examples of such rivers.

* Called the point of centra flexure.

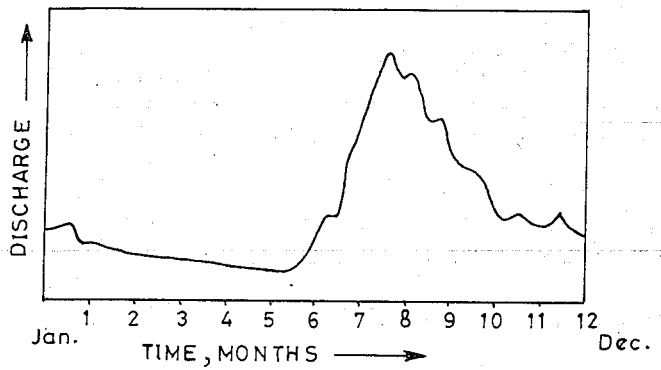


Fig. 7.47. Annual hydrograph of a perennial stream.

Intermitten streams flow only during the monsoon season (Fig. 7.48), and hence carry base flow contribution only during the rainy season. The watertable in the area is above the river bed only during rainy season. Minor & medium rivers of Indian Peninsula mostly fall under this category.

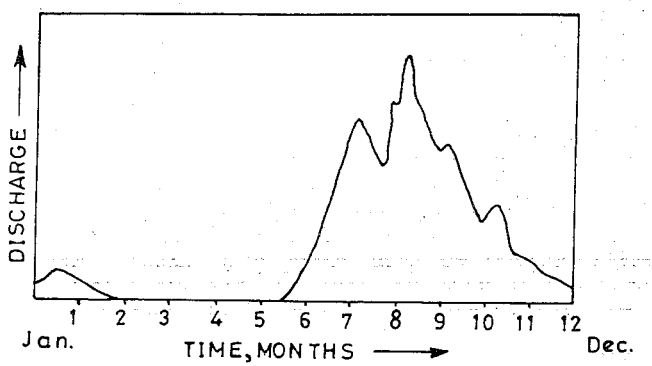


Fig. 7.48. Annual hydrograph of an intermittent stream.

Ephemeral streams, on the other hand, do not receive any base flow contribution, since the watertable in the area is always lower than the stream bed, all along. The hydrograph of such a stream (Fig. 7.49) exhibits flash flows in response to rain storms. Luni river of Rajasthan State in India is an example of this type of river.

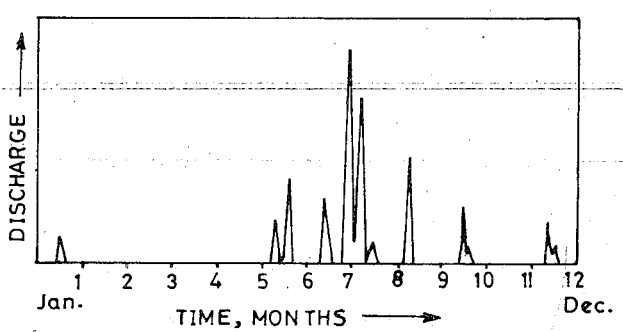


Fig. 7.49. Annual hydrograph of an ephemeral stream.

ABSTRACTIONS FROM RAINFALL INCLUDING INFILTRATION AND EVAPORATION

Water is constantly evaporated from the earth, and is precipitated back on the earth, mainly in the form of rainfall. One part of this rainfall sinks into the ground, forming ground water reservoir; second major part flows as runoff in the form of rivers; and the rest is lost in evaporation and transpiration. The computations of infiltration, evaporation, transpiration and run off are discussed in the subsequent articles.

7.33. Infiltration

When water falls on a given formation, a small part of it is first of all, absorbed by the top thin layer of soil, so as to replenish the *soil moisture deficiency* (explained later). Thereafter, excess water moves downward, where it is trapped in the voids, and becomes *ground water.*

This process, whereby the water enters the surface strata of the soil, and moves downward towards the watertable, is known as *infiltration.*

7.33.1. Infiltration Capacity. The ground water stored in the under-ground soil depends mainly upon the number of voids present in the soil, which, in turn, does not depend upon the size of the soil particles but rather upon the arrangement, sorting, shape and degree of compaction. Therefore, different soils will have different number of voids, and hence, different capacities to absorb water. *The maximum rate at which a soil in any given condition is capable of absorbing water is called its infiltration capacity.* It is generally denoted by the letter *f.*

7.33.2. Infiltration Rate. It is evident that the rain water will enter the ground soil at full capacity rate (f) only during the periods when rainfall rate exceeds the infiltration capacity. When the rain intensity is less than the infiltration capacity, the prevailing infiltration rate is approximately equal to the rainfall rate. Hence, the actual prevailing infiltration rate may be equal to or less than the infiltration capacity. This actually prevailing rate at which the water will enter the given soil at any given time is known as the *infiltration rate.*

If the rain intensity (p) exceeds the infiltration capacity (f), the difference is called the *excess rainfall rate* (p_e). This excess water is, first of all, accumulated on the ground as surface detention (D), and then flows overland into the streams.

7.33.3. Soil Moisture. The water below the watertable is known as the *ground water,* and the water above the watertable is known as the *soil moisture.*

The region above the watertable is divided into three zones:

1. Capillary zone ;
2. Intermediate zone; and
3. Soil zone.

Extending above the watertable, a distance usually ranging from about 0.3 to 3 m, depending principally upon texture, is a zone, called the capillary zone or the *capillary fringe.* Throughout this capillary zone, the moisture content is maintained practically constant by capillarity.

Extending down from the ground surface, is the *soil zone,* which is defined as being the depth of overburden that is penetrated by the roots of vegetation. Throughout this *root zone,* the moisture content varies tremendously, ranging from a partly saturated state during and immediately following the periods of heavy rains, to a minimum content after a spell of long continued drought.

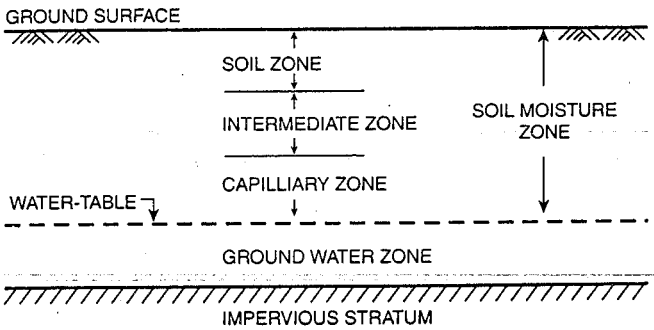


Fig. 7.50

The region between the capillary zone and the soil zone is called the *intermediate zone.* Throughout this zone, except during the periods of ground water accretion from rainfall, the amount of water contained within any given space is nearly constant throughout the year. In some cases, the height of capillary fringe (or capillary zone), may be more, and may extend upto the soil zone. In that case, there will be no intermediate zone. Various zones are shown in Fig. 7.50.

7.33.4. Field Capacity. Immediately after a rain, when all the gravity water has drained down to the watertable, a certain amount of water is retained on the surfaces of the soil grains by molecular attraction. This is known as *pellicular water.* This water cannot be easily removed. The degree of its resistance to the movement is expressed by the surface tension. *When this water, which any soil can retain indefinitely against the force of gravity, is expressed in term of depth of water (as if it were spread on the basin), then it is known as the field capacity.*

A certain portion of this pellicular water which can be easily abstracted by the root action of vegetation is called the *available moisture.* The remainder is unavailable moisture, and is known as the *hygroscopic water.* The plants can extract water from the soil till the wilting-point is reached. Therefore, the available moisture is the moisture between the field capacity (upper limit) and the wilting point (lower limit). The wilting point is that moisture content at which permanent wilting of plants take place. The hygroscopic water is therefore, the moisture content of the soil, just after the soil has been wilted up.

The depth of the water required to bring the soil moisture content of a given soil upto its field capacity, is called the *field moisture deficiency or soil moisture deficiency.*

Whenever a rain falls, it must, first of all, supply and fill up the soil moisture deficiency, and then only there can be any ground water accretion. However, the soil moisture deficiency will be different at different points, and the falling rain will replenish it at different rates at different points. Hence, in the same drainage basin, it is very much possible that the ground water accretion starts at one point, while the soil moisture deficiency still exists at some other point.

7.33.5. Equivalent Moisture. Just as the *field capacity* is the water retained by a saturated soil after being acted upon by gravity, similarly the *equivalent moisture* is the water retained by a saturated soil after being centrifuged by a centrifugal force of 1000 times that of gravity. Therefore, it is slightly less or at the most equal to the field capacity.

7.33.6. Factors Affecting Infiltration Capacity. The capability of a formation to absorb water is known as its infiltration capacity (f). It is not a constant factor. It is not only changes with time but also changes with location. The value of f at any given time and a given location is the resultant effect of the interaction of various factors. Some of these factors change the value of f with the change of location, and others change it from time to time at a given section. There are certain other factors like the vegetative cover, which may change the value of f with both location and time. The various factors which affect the value of f are described below:

(1) **Thickness of the Saturated Layer and the Depth of Surface Detention.** The water infiltrates into the ground under the force of gravity, and there exists a layer of soil near the surface having its interstitial spaces saturated. If the thickness of this saturated soil at any given time at a given section is l, then the water will flow through a series of tiny tubes of length l, as shown in Fig. 7.51.

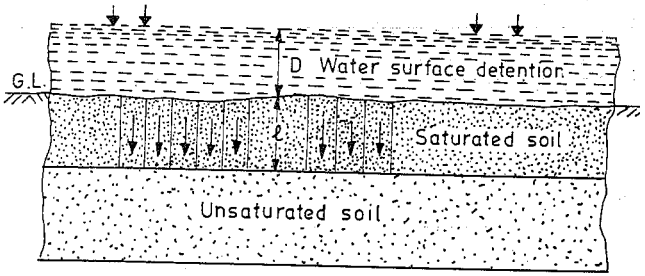


Fig. 7.51

At the top of each tube, the pressure head is equal to D (*i.e.* equal to the surface detention) and total pressure head causing discharge is $D + l$. While on the other hand, the resistance to the flow which is offered by the soil is proportional to l.

Hence; Force (F) causing discharge $\alpha (D + l)$ ...(7.34)

and Resistance to the flow (R) $\alpha\, l$ ...(7.35)

If l is large as compared to D, changes in l will have nearly equal effect on the force and the resistance, and the rate of infiltration is nearly constant.

However at the beginning of a rain, D and l may be of the same order of magnitude. Under such a condition, force is large compared with the resistance, and water will enter the soil rapidly. But with the passage of time, l will become much more than D and, therefore, there will not be much difference between the values of F and R, and hence, the rate of infiltration falls down. This is one of the reasons as to why f is relatively large at the beginning of a rain.

(2) **Soil Moisture.** The amount of the soil moisture has an important effect on the infiltration capacity. The effects of soil moisture on f are two folds, as described below:

(*a*) In early winter or spring, the soil moisture content is generally high, and the value of f is low; while in summer season, the soil moisture content is low, and so the value of f goes up. When the soil is dry, the infiltration is more, because of the following reason :

When water falls on a dry soil, the upper surface becomes wet, while the lower parts of this soil remain comparatively much dry. Thus, there exists a large difference of capillary potential between the top surfaces of the soil and those below it. Due to this difference of capillary potential, a downward force will act on the water which will be

in addition to the normal force of gravity. Hence, the water will now move downward with two forces, and hence rapidly. As the time passes, the lower surface of soil will also go on becoming wet, and this difference of capillary potential will go on reducing, and hence, the infiltration will go on falling, as the soil moisture content goes on increasing.

(*b*) The second effect of the soil moisture on f is the reverse of the first. When the soil becomes wet, the colloids present in the soil swell immediately, which, reduces the infiltration capacity during the initial period of rainfall. This is one of the reasons as to why there is a rapid decrease in f during a rain. Hence, this factor is responsible for much of the typical seasonal variation in the value of f as well as a part of the rapid decrease in f during a rain.

(3) **Compaction due to Rain.** When rain falls over the soil, the soil automatically undergoes mechanical compaction. This compaction reduces the voids in fine textured soils like clay, and hence, reduces their infiltration capacity. A lot of effect can be produced in this manner on the exposed clay soils, which on compaction, become virtually impermeable; whereas, the infiltration capacity of a clean sandy soil is affected very little by rain compaction.

This is another factor which causes the infiltration capacity to decrease rapidly during the initial portion of a rain storm.

(4) **Washing of Fines.** When a soil becomes very dry, the surface often contains many fine particles. When the rain falls, and the infiltration begins, these fines are taken down into the soil, and are deposited in the voids, thus reducing the infiltration capacity. This factor also reduces f during a rain.

(5) **Compaction due to Movement of Men and Animals.** When there is a heavy live load moving on the surface, the soil gets compacted, and hence rendered relatively impervious, thereby reducing the value of f. Playgrounds, pagdandies etc. are the examples of this type of soil.

(6) **Vegetative Cover.** This factor is related to many of the factors described above, but this factor is the most important factor and it will not be unwise to discuss its effect under a separate head.

The pressure of a dense vegetative cover over a soil, increases the infiltration capacity of that soil to a considerable extent. In the presence of a vegetative cover, the rain will not fall directly on the soil, and hence not able to compact the soil. The vegetation cover also provides a layer of decaying organic matter, which promotes the activity of burrowing insects and animals, which in turn, produces spongy permeable soil structure. Both these factors help in increasing the infiltration capacity, and hence the presence of a vegetative cover causes the infiltration capacity to increase predominantly. So much so, that the infiltration capacity of a soil protected with a vegetation cover may be much larger than the infiltration capacity of the barren soil.

Further, transpiration by vegetation removes soil moisture, and thus tends to increase f during initial periods of rain. Open crops, such as potatoes or corn, provide little protection from rain compaction, and serve only in producing a partial cover of organic matter, with the result that values of f may be relatively low in areas covered by such crops.

(7) **Temperature.** The viscosity of water changes with temperature. Since the flow in the interstitial spaces of soil is nearly always laminar, the rate of infiltration will also change with viscosity. Hence f will change with temperature. This factor causes the value of f to be somewhat lower in winter and somewhat higher in summer.

7.33.7. Infiltration Capacity Curve (I.C. Curve). The infiltration capacity curve is the graphical representation of the variation of infiltration capacity with time, during and a little after rain.

As has been said earlier, the infiltration capacity is generally very high at the beginning of a rain storm that occurs after a long dry period. During the rain storm, the infiltration capacity is considerably reduced due to the various factors, such as, surface detention, soil moisture, compaction due to rain, washing of fines, etc. as described earlier. After a certain period (of the order of 1 to 3 hours) the infiltration capacity tends to become constant.

A typical infiltration capacity curve is shown in Fig. 7.52.

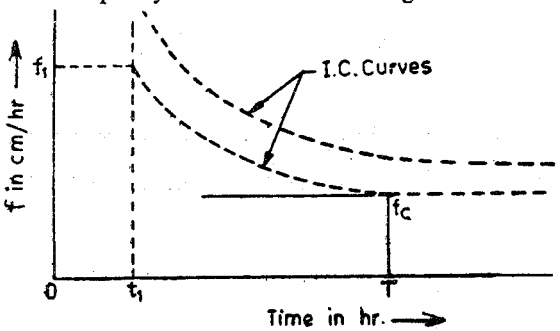


Fig. 7.52. Typical infiltration capacity curves.

where, O = Origin-the time is counted from the start of the rain.

t_1 = Time during which there is no infiltration, and the entire rain is used up in interception and depression storage.

Sometimes, the value of f is to be used for computing the surface runoff due to a given storm from a given drainage basin; in that case, the value of f is first of all calculated from another known storm. This value of f is so calculated as to include interception and depression storage in itself, and it is a general practice to include interception and depression storage in the infiltration itself. The curve will then be represented as shown in Fig. 7.53. This is known as *standard infiltration capacity curve.*

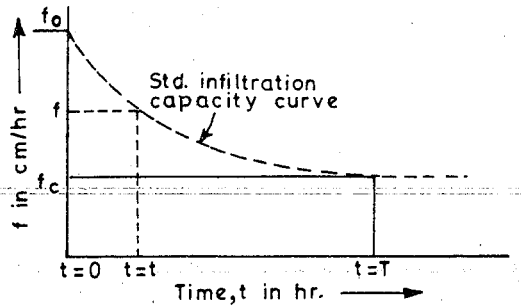


Fig. 7.53. Standard infiltration capacity curve.

where, f_0 = value of infiltration capacity at $t = 0$

f_c = constant value of infiltration reached towards the end.

The value of f_0 may be twice, thrice or any other times of f_c. For different basins, this ratio varies widely depending upon the amount of interception, depression storage, and upon the type, texture and condition of the soil.

7.33.8. Equation of I.C. Curve. Since this curve is reaching a constant value and, therefore, it is a curve of the exhaustion type. Horton has suggested a mathematical form by which this curve is generally represented, as given below:

$$f = f_c + (f_0 - f_c)\, e^{-Kt} \qquad ...(7.36)$$

where f = Infiltration capacity at any time t.
f_c = The value of infiltration rate after it reaches a constant value.
f_0 = Infiltration capacity at the start
K = a constant.
t = Time from the beginning of rainfall.

In order to solve the above equation, let us proceed further as follows:

$$f = f_c + (f_0 - f_c)\, e^{-Kt}$$

Transposing, $\quad f - f_c = (f_0 - f_c)\, e^{-Kt}$

Taking log on both sides

$$\log_{10}(f - f_c) = \log_{10}(f_0 - f_c) - Kt \log_{10}^{e}$$

or $$\log_{10}(f - f_c) - \log_{10}(f_0 - f_c) = -Kt \log_{10}^{e}$$

$$\therefore \quad t = \frac{-1}{K \log_{10}^{e}} \left[\log_{10}(f - f_c) - \log_{10}(f_0 - f_c)\right]$$

or $$t = \frac{-1}{K \log_{10}^{e}} \log_{10}(f - f_c) + \frac{1}{K \log_{10}^{e}} \log_{10}(f_0 - f_c)$$

The above equation is of the form

$$Y = mX + C$$

where, $Y = t$,

$$m = \frac{-1}{K \log_{10}^{e}} \qquad ...(7.37)$$

$$X = \log_{10}(f - f_c)\ ; \text{ and}$$

$$C = \frac{1}{K \log_{10}^{e}} \log_{10}(f_0 - f_c)$$

and hence, represents a straight line, having a slope $= \dfrac{-1}{K \log_{10}^{e}}$

The –ve sign shows that as t increases, f decreases, and, therefore, $(f - f_c)$ decreases, and hence log $(f - f_c)$ decreases. This straight line graph is of the form shown in Fig. 7.54.

If two values of f at two known times are known and f_c is also known, the straight line can be drawn through these two points, and the slope of the line measured; and then equating it to $\dfrac{-1}{K \log_{10}^{e}}$, K can be determined, and hence, an equation for the I.C. curve can be written easily.

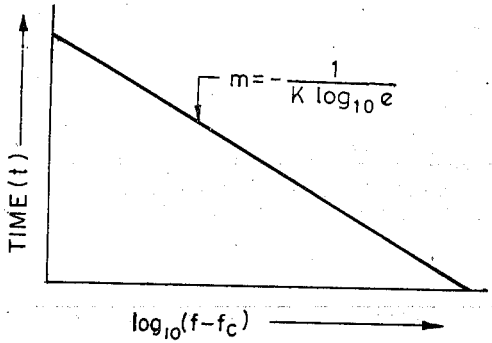


Fig. 7.54

Example 7.13. *The infiltration capacities of an area at different intervals of time are indicated below. Find an equation for the infiltration capacity in the exponential form.*

Time in hours	*0*	*0.25*	*0.50*	*0.75*	*1.00*	*1.25*	*1.50*	*1.75*	*2.00*
Infiltration capacity (f) in cm/hr	*10.4*	*5.6*	*3.2*	*2.1*	*1.5*	*1.2*	*1.1*	*1.0*	*1.0*

(U.P.S.C. Engg. Services, 1970)

Solution. Let the equation for the I.C. curve be, $f = f_c + (f_0 - f_c)\, e^{-Kt}$

In this question, the infiltration capacity reaches a constant value equal to $f_c = 1.0$.

Now, let us tabulate the values of $(f - f_c)$ and $\log_{10}(f - f_c)$ for various values of t, as shown below in table 7.22.

Table 7.22

t in hours	0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
f in cm/hr	10.4	5.6	3.2	2.1	1.5	1.2	1.1	1.0	1.0
$f - f_c$	9.4	4.6	2.2	1.1	0.5	0.2	0.1	0.0	0.0
$\log_{10}(f - f_c)$	0.973 or +0.973	0.663 or +0.663	0.342 or +0.342	0.042 or +0.042	$\bar{1}$.699 or −0.301	$\bar{1}$.301 or −0.699	$\bar{1}$.00 or −1.00		

Now, plot a graph between t and $\log_{10}(f - f_c)$*, and pass a straight line through the plotted points, as shown in Fig. 7.55.

Measure the slope of this line from the graph. It is equal to

$$-\frac{0.765}{1} = -\frac{1}{1.31}$$

But the slope of this line, when its equation is

$$f = f_c + (f_0 - f_c)\, e^{-Kt}$$

is represented by $-\dfrac{1}{K \log_{10} e}$ $\quad\therefore\quad$ $-\dfrac{1}{K \log_{10} e} = -\dfrac{1}{1.31}$

or $\quad 1.31 = K \log_{10} 2.718 = K \times 0.4343$

or $\quad K = \dfrac{1.31}{0.4343} = 3.02$

* If a semi-log graph is available, the values of t on ordinary scale and those of $(f - f_c)$ on log scale can be directly plotted, thus, avoiding computing values of $\log_{10}(f - f_c)$.

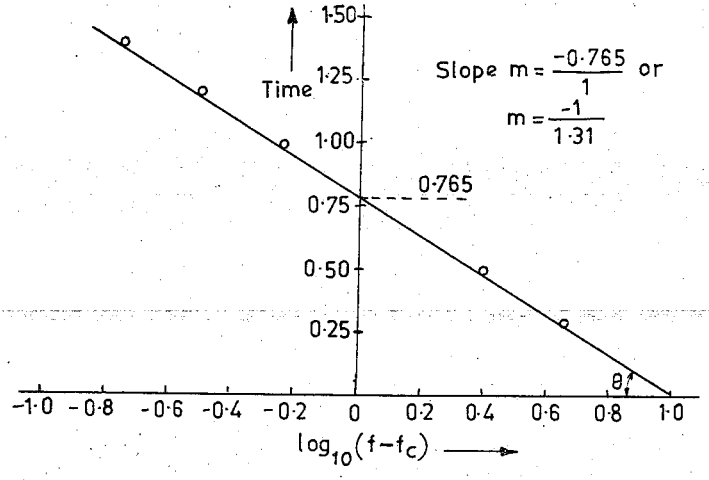


Fig. 7.55

Hence, the equation for I.C.C. is given by

$$f = 1.0 + (10.4 - 1.0)\, e^{-3.02t}$$

or $$f = 1.0 + 9.4\, e^{-3.02t}$$ **Ans.**

where f is in cm/hr and t is in hours.

Note. The equation of I.C.C. of a given catchment can be further used to determine the total depth of infiltration for a storm of the given duration, as exhibited in the Example below :

Example 7.13. (*a*) *Assuming the initial infiltration rate of 10 mm/h, final infiltration rate of 5 mm/h and the constant value (describing the rate of decay of the difference between the initial and final infiltration rates) as 0.95 h^{-1}, calculate the total infiltration depth for a storm lasting 6 h.* (*Civil Services, 2004*)

[**Solution.** f_o = Initial infiltration capacity/rate = 10 mm/h.

f_c = Final constant infiltration capacity/rate = 5 mm/h

f = Infiltration rate at any time t.

The equation of infiltration capacity curve is given by Eq. 7.36 as :

$$f = f_c + (f_o - f_c)\, e^{-Kt}$$

where K is a constant having unit of t^{-1}

$= 0.95\ (\text{hour}^{-1})$

Substituting values, we get

$$f = 5 + (10 - 5)\, e^{-0.95t}$$ when t is in hr.

To find the total infiltration Σf in mm during 6 hr storm, we have to integrate the above equation of ICC between 0 hr to 6 hr, as

$$\sum_{0}^{6\text{ hr}} f = \int_{t=0}^{t=6} 5 + 5\,.\,e^{-0.95t}$$

$$= \int_{t=0}^{t=6} 5 + 5 \int_{t=0}^{t=6} e^{-0.95t}$$

$$= 5 \Big| t \Big|_0^6 + 5 \left[\frac{e^{-0.95t}}{-0.95} + c \right]_{t=0}^{t=6}$$

$$= \left[5 \times 6 - 0 \right] + 5 \left[- \left\{ \frac{e^{-0.95 \times 6}}{0.95} - \frac{e^{0}}{0.95} \right\} \right] + 5 \left| c \right|_{t=0}^{t=6}$$

$$= 30 + \frac{5}{0.95} \left[e^{0} - e^{-5.70} \right] + 5 \left[c - c \right]$$

$$= 30 + \frac{5}{0.95} \left[1 - 3.346 \times 10^{-3} \right]$$

$$= 30 + 5.25 = \mathbf{35.25\ mm.\ Ans.}$$

7.33.9. Methods of computing Infiltration Capacity. Two methods are generally used to determine infiltration capacity:

1. Experimental methods with the use of infiltrometers or rain-simulators.
2. By analysing the rainfall hyetograph and runoff hydrograph.

The first method consists in applying water artificially on the soil, whose infiltration capacity is to be determined, and, then observing and analysing the actual infiltration. This method is not of much importance for determining the infiltration capacity value which has to be reused for computing the runoff from the same or a similar basin. This is because of the fact that the results obtained by the use of Infiltrometers are qualitative and are not quantitative. In other words, the method is useful in determining the relative effects of any change in land use, slope, vegetative cover, etc. The numerical values of f obtained by this method may be quite erroneous. Therefore; in order to obtain more precise value of f, which can be reused for calculating runoff, the second method is generally used.

Before describing these two methods, we will, first of all, differentiate between two important terms; namely (*i*) Small watersheds; and (*ii*) Large watersheds.

(*i*) **Small Watersheds.** Small watersheds are those drainage basins which are small in size to such an extent that the rain intensity may be considered as being uniform over the entire basin. The area of such a basin may range from a few hectares to perhaps *2500 hectares*. Such a basin will also respond quickly to rainfall, and hence, each period of intense rainfall is likely to produce a separate peak in the runoff hydrograph. Such basins are generally encountered in the design of culverts, storm sewers, small bridges, etc.

(*ii*) **Large Watersheds.** The large drainage basins are the basins of longer dimensions, and, thus, larger than those for which the rain intensity may be considered as being uniform over the entire basin. They are generally involved in the construction of flood protection works (such as dams, etc.), irrigation works, water supply works, etc.

(1) Determination of I.C. by Using Infiltrometers and Rain Simulators. Various kinds of infiltrometers are used to measure f. The most common type consists of two shallow concentric rings of sheet metal, as shown in Fig. 7.56. It is placed on the ground

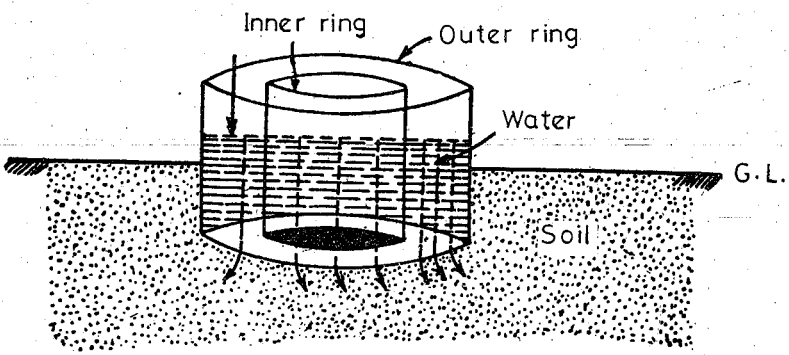


Fig. 7.56. A typical infiltrometer.

with the upper portion projecting above the ground, and lower portion lying under the ground. Water is filled in both the compartments and kept at the same level.

The outer ring will prevent the water of the inner ring from spreading over a large area after penetrating below the bottom of the ring. The rate at which water is required to be added to the inner ring, in order to keep constant level, will directly give us an idea of the infiltration capacity.

To be more close to the actual conditions, sometimes artificial rain test is conducted. The instrument used for artificial rain is called a *rain simulator.* With the help of a simulator, water is sprinkled at a uniform rate in excess of the infiltration capacity, over a certain experimental area. The resultant runoff pattern is observed, and from that "*f* curve" is derived. This *f* curve can then be reused to determine runoff from the basin, on which the test was conducted.

(2) **Determination of I.C. Curve for Small Drainage Basins (Horner and Llyod's Method).** This method consists in analysing the rain intensity pattern and the resulting runoff hydrograph. Small drainage basins are quick to respond to rain intensity, and, therefore, each period of intense rainfall is likely to produce a separate peak in the resulting runoff hydrograph. The difference between the rainfall during a certain interval of time and the resultant runoff will give us the amount of infiltration (F) during that interval. The quantity F is then divided by the time during which this infiltration has taken place, so as to obtain the value of f during that interval.

Mathematically, we can write

$$F = P - Q$$

$$\therefore \qquad f = \frac{F}{t} = \frac{P - Q}{t} \qquad \ldots(7.38)$$

where P = Total rain

Q = Total Runoff

t = Time during which the infiltration has taken place.

We also know that excess rain appears as surface runoff, but only after some delay. In other words, there exists a lag between the time when rainfall occurs, and the time when that water appears as surface runoff at the outlet of the basin. Therefore, the infiltration starts at the beginning of excess rainfall and continues, not only upto the end of rain but for some more time even after the rain ceases. Just at the end of excess rainfall, the infiltration takes place from the full area, but afterwards, the area goes on reducing. In other words, the area of infiltration goes on reducing constantly during this extra period. For small drainage basins, Horton assumed that the infiltration, which occurs during this extra period, is equivalent to an amount of infiltration occurring over the entire area for a period equal to one-third of this extra period (*i.e.* the period that elapses from the end of the excess rainfall until the end of overland flow). Horton also assumed that this overland flow stops at the point of inflexion of the hydrograph.

From the rain intensity pattern, the time when excess rain ceases, is known. And from the point of inflexion of the runoff hydrograph, the time when overland flow ceases, is also known. The difference between these two times will give us this extra period, 1/3rd of which when added to the excess rainfall period, will give us the total time of infiltration (t) to be used in equation (7.38) above.

Values of f are computed in this manner for various intense rainfall intervals, and plotted at a distance $t/2$ after the beginning of excess rainfall periods. A curve is then drawn through these points, so as to obtain I.C. curve. The method will become more clear on solving the following example.

Example 7.14. *The rainfall rates on a small drainage basin were observed during a storm of 145 minutes duration and are tabulated below.*

Time from Start in minutes	*20*	*40*	*60*	*80*	*100*	*120*	*140*	*145*
Rainfall rate in cm/hr.	*2.0*	*10.0*	*7.5*	*1.5*	*1.5*	*5.0*	*1.0*	*0.0*

The resultant runoff at the outlet of the basin was observed and a hydrograph plotted. The hydrograph of this runoff is found to have two separate peaks of the type shown below in Fig. 7.57. The two segments (A) and (B) of the hydrograph are found to contain an area of 2.3 cm and 1.0 cm depth of water, respectively. The point of inflexion of the segments were found to be at t = 84 min, and t = 149 min., respectively.

From these observed rainfall runoff data, compute the infiltration capacity curve and plot it on the rain hyetograph.

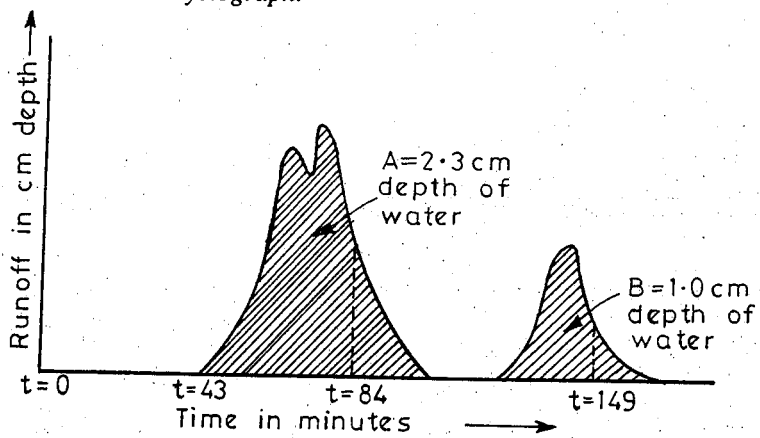


Fig. 7.57

Solution. First of all, let us plot the rain hyetograph from the observed rainfall rates, as shown in Fig. 7.58 (*a*).

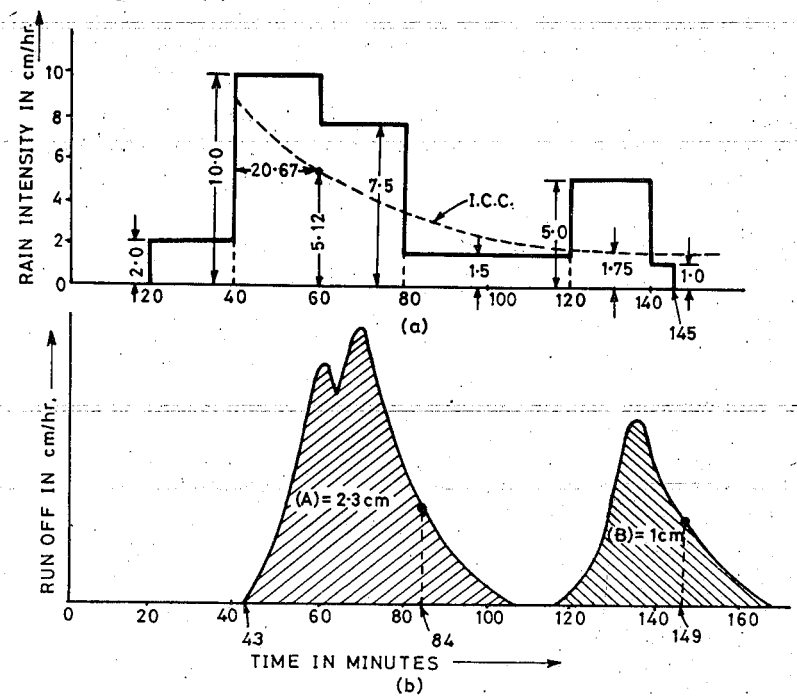


Fig. 7.58 (*a*) and (*b*)

The observed hydrograph of runoff is also plotted below the rain hyetograph, as shown in Fig. 7.58 (*b*).

It is evident from the rainfall hyetograph that there are two periods of intense rainfall (*i.e.* from $t = 40$ to $t = 80$, and $t = 120$ to $t = 140$ min.) which can be designated as periods of excess rainfall. Let us designate them as period *a* and period *b*. The amount of rain during the period *a*

$$= P_a = \left(10.0 \times \frac{20}{60} + 7.5 \times \frac{20}{60}\right) = \frac{17.5}{3} = 5.83 \text{ cm}$$

The amount of rain during the period $b = P_b = 5.0 \times \frac{20}{60} = 1.67$ cm

Q_a = Runoff during the period *a* = Area under the graph $A = 2.3$ cm (given)

Q_b = Runoff during the period *b* = Area under the graph $B = 1.0$ cm (given)

F_a = Infiltration during the period $a = P_a - Q_a = 5.83 - 2.3 = 3.53$ cm

F_b = Infiltration during the period $b = P_b - Q_b = 1.67 - 1.0 = 0.67$ cm

Now, let us determine the time of infiltration for these two periods.

$$t_a = \text{Excess rainfall period} + \frac{\text{Extra time}}{3}$$

$$= 40 \text{ min.} + \frac{84 - 80}{3} \text{ min.} = 41.33 \text{ min.} = \frac{41.33}{60} \text{ hr.}$$

$\because$ $t = 84$ min. is the point of inflexion; and
$t = 80$ min. is the time at the end of excess rainfall interval.

Similarly, $t_b = 20 \text{ min.} + \frac{149 - 140}{3}$ min.

$$= 23 \text{ minutes} = \frac{23}{60} \text{ hours}$$

Now $$f_a = \frac{F_a}{t_a} = \frac{3.53}{\frac{41.33}{60}} = \frac{3.53 \times 60}{41.33} = 5.12 \text{ cm/hr}$$

and $$f_b = \frac{F_b}{t_b} = \frac{0.67}{\frac{23}{60}} = \frac{0.67 \times 60}{23} = 1.75 \text{ cm/hr.}$$

These two points f_a and f_b are now plotted at distances $\frac{t_a}{2}$ *i.e.* 20.67 min, and $\frac{t_b}{2}$ *i.e.* $\frac{23}{2} = 11.5$ min, respectively from two periods of excess rainfall on the rain hyetograph. A smooth curve is plotted through these two points, which is extended back and far, so as obtain the required I.C. curve; as shown by the dotted line in Fig. 7.58 (*a*).

(3) **Determination of Average I.C. (ϕ_{index}) for Large Drainage Basins (Horton's Method).** For large drainage basins, the rain intensity cannot be considered uniform over the entire area, and the runoff does not respond quickly to the rainfall pattern. Therefore, the previous method described to determine I.C. curve for small basins is not applicable to large basins. Horton has developed a method for determining the infiltration capacity of large drainage basins. It is not possible to determine I.C. curve or instantaneous values of infiltration capacity for large basins. Only the average values can be determined for their infiltration capacities.

Assumptions

The various assumptions made by Horton in computing I.C. for large drainage basins are enumerated below :

(1) For most of the storms responsible for producing major floods on large basins, the rain intensity patterns at adjacent stations are very similar.

(2) The surface runoff is equal to the difference between the rainfall and the infiltration that occurs during the period of rainfall excess. In other words, the rain that falls during and immediately following the periods of rainfall excess, but infiltrates during the subsequent period of overland flow, is ignored. Even after making these two assumptions, the difference between total rainfall (P) and total runoff (Q) cannot be divided by the period of rainfall excess shown by the recording gauge; because, one must not forget that the period of rainfall excess varies throughout the basin and is different at various stations.

Procedure. Here we will describe a method, which is used to determine a value of infiltration capacity (ϕ_{index}) which when multiplied by the period of rainfall excess (t_e) will give us the difference in rainfall and runoff during that period. Mathematically,

$$P - Q = t_e \, \phi_{index}$$

or

$$Q = P - t_e \cdot \phi_{index}$$

where P = Total rainfall for the period t_e.

Q = Total runoff for the same period

ϕ_{index} = Av. value of I.C. (explained in details in article 7.43).

Thus, for large basins, only average values of I.C., *i.e.* ϕ_{index} can be worked out.

The large drainage basin is divided into a number of stations. One of them must be fitted with an automatic recording gauge, so that the hourly rainfall values at this station are known. This is known as the base station. The daily rainfall records at the other stations (for the same period) are also collected and adjusted to a common 24 hour period. Let a basin be divided into six stations and their 24 hour rainfall values be given below.

Station	*24 hr. rainfall in cm*
A	15.0
B	9.0
C	8.5
D	10.5
E	10.7
F	15.9
	Average = 11.6 cm.

The hourly recorded rain values at the base station are, first of all, converted as percentages of the total. Then a total rainfall of some whole number of cm (≈ the average depth for the given storm) is assumed. The hourly rain values at the base station due to this assumed whole number rain (such as 10 cm) are then worked out. Various values of infiltration capacity are assumed, and the hourly rainfall excesses are worked out. Their summation will give us the total rainfall excess for an assumed value of infiltration capacity. Various such values for various values of infiltration capacity are worked out for a given rainfall (10 cm).

Various other whole number values of total rainfall, such as 6, 8, 12, 14, 16 cm are assumed, and process repeated. Every time a table is drawn.

A graph is then plotted between total rainfall values (such as 6, 8, 10, 12, 14, 16 cm. etc.) on *X*-axis and corresponding rainfall excess on *Y*-axis for different values of infiltration capacity. Thus, different curves can be obtained for different values of infiltration capacity, as shown in Fig. 7.59.

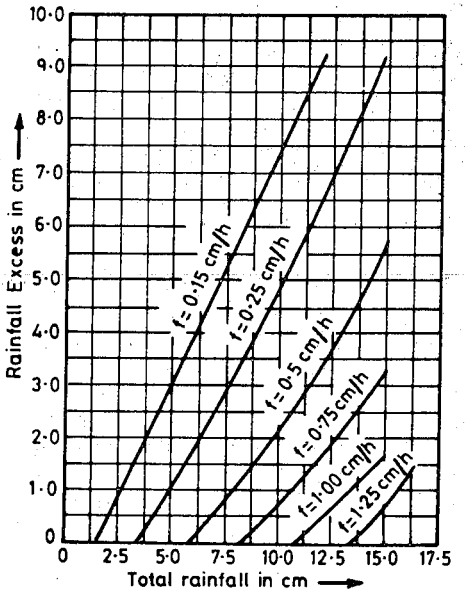


Fig. 7.59

The 24 hr. rainfall values of various stations such as 15 cm, 9.0 cm, 8.5 cm, ... are now used to read out from Fig. 7.59, the values of rainfall excess for various infiltration capacities such as 0.25, 0.50, 0.75, ... cm/h. The rainfall excess values of all the stations are then averaged to find single rainfall excess value for each value of infiltration capacity. All such excess rainfall values and their corresponding infiltration capacity values are finally plotted, as shown in Fig. 7.60, to obtain the requisite relationship between rainfall excess and average infiltration capacity of the basin.

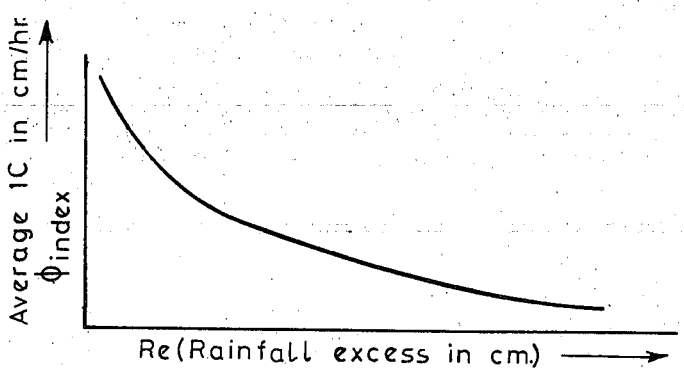


Fig. 7.60

7.34. Evaporation and Transpiration

We have already stated that the precipitation occurring over a drainage basin is partly lost in interception, ground depression, and in replenishing soil-moisture deficiency, before it contributes to the accretion of ground water and runoff. The water which is intercepted by objects, and the water of ground depressions which does not later infiltrate to ground water reservoir, evaporate back to the atmosphere, as vapour. The water used in replenishing soil moisture deficiency is also evaporated back to the atmosphere, either through direct land evaporation or through plants, which continuously release water into the atmosphere through their breathing or metabolic process, called *transpiration.*

When short time periods are considered, such as the duration periods of rainfall or runoff, then no water from the ground reservoir may be lost to the atmosphere; but however, when longer periods are considered, then ground water extracted by plants through capillary fringe, may also be lost through transpiration.

Therefore, while considering short time events from a drainage basin, such as the evolution of huge run off and flood waters from a basin, we generally consider only the infiltration, and neglect the *initial basin losses* that occur during the rain period. However, when long term events from a drainage basin are desired, such as the computation of the monthly or annual or even longer time yields from the drainage basin, then eventually, we cannot ignore the continuously occurring evaporation and transpiration losses in the drainage basin. It is, therefore, necessary to understand the mechanism behind such losses, and methods used to estimate them.

As a matter of fact, the water lost through evaporation and transpiration from all the basins of a region (including lakes, oceans, etc.), constitutes a very important part of the hydrologic cycle of that region, because it is this process by which the precipitation coming to the region returns to the atmosphere for reprecipitation, thereby maintaining the hydrologic cycle.

Although all the losses from a basin are strictly evaporative in character, yet they can be classified into the following four types:

(i) interception losses;

(ii) evaporation losses from water surfaces;

(iii) transpiration losses; and

(iv) land evaporation losses.

These losses have been discussed below to understand each one of them, and, thus, to help in the final estimation of the total loss, caused by the factors applicable in the given basin.

7.34.1. Interception Losses. When a rain falls, it is first of all intercepted rapidly by trees, plants, buildings, etc., till they become completely wet. The interception rate (generally expressed in mm/h or cm/h) is very high in the beginning of the rain, but goes on reducing as the rain continues. When the surfaces become thoroughly wet, the interception rate falls down, and becomes equal to the evaporation rate from those surfaces for the rest of the rain period. The amount of water required to make these surfaces completely wet is known as *interception storage capacity* and may be denoted by a. The total interception on any drainage basin for a given condition of vegetation may be represented by the equation,

$$x = a + bt \qquad ...(7.39)$$

where x = Total interception in cm

a = Interception storage capacity in cm

b = Evaporation rate from the intercepting surfaces in cm/hr

t = Duration of showers in hr.

It is evident from the above equation that the interception depends mainly upon the duration of the rainfall, and also upon the frequency, but very less upon the intensity of the rainfall. Hence, if the total rainfall is more, the interception may be only a little more. Thus, the interception, expressed as percentage of the total rainfall, reduces, as

the total precipitation increases. For example, if the interception is 3 cm in a total rainfall of 4 cm (equal to 75%), it may rise to 3.5 cm in a total rainfall of 10 cm (equal to 35% only), thereby reducing its percentage from 75% to 35%.

7.34.2. Evaporation Losses from Water Surfaces. The liquids are constantly changed into vapour through the process of evaporation. The exact laws governing this process are not clearly understood, but the most commonly adopted theory behind the process of evaporation is given below:

As is true for other bodies, water also consists of large number of molecules, which are constantly moving with different velocities in different directions. Their average velocity represents the temperature of water. At temperature of zero degree absolute, the velocity is supposed to be zero, *i.e.*, the molecular activity is supposed to cease.

Each of these molecules is attracted to every other molecule by a force directly proportional to the product of their masses and inversely proportional to the square of the distance between them. Thus, a molecule of water, which lies near the surface, will have more forces attracting it from below than from above. Therefore, this molecule is less rigidly held and can be detached from the water surface, if its own velocity is sufficient to overcome the resisting force. It, therefore, appears that only the fast moving molecules can escape into air as vapour. And when a fast moving molecule goes out of the body of water, the average velocity of the remaining molecules will fall down, and thus, reducing the temperature of water. This is the reason why cooling is produced by evaporation.

The molecules of outermost boundary layer, thus, continuously leave the water surface, and others do come to take their place. But the number of leaving molecules, is not equal to the number of arriving molecules. If the number of leaving molecules becomes more than the number of returning molecules, the difference gives the rate of evaporation. If the process is reversed, *i.e.*, if the number of returning molecules becomes more than the number of leaving molecules, condensation will take place.

Moreover, in the immediate vicinity of the water surface, a thin film of air does exist. The temperature of this film of air is the same as that of water. When water vapour come out of a given body of water, they try to crowd this film of air, and thereby quickly saturating the same with water vapour. The vapour pressure of this film of air, when it is fully saturated with vapour, is known as *saturation vapour pressure*. If the actual pressure of the air above this thin saturated film is less than the saturation vapour pressure, the vapour already collected in this film will disperse into the air and the evaporation will continue; but if the actual vapour pressure of air is more than the saturation vapour pressure, then there can be no evaporation. The rate of evaporation is thus directly dependent upon the difference of these two pressures.

7.34.2.1. Dalton's law of evaporation. The above principle is applied in Dalton's law of evaporation, which is generally expressed by the following equation:

Evaporation (E) $\propto (e_s - e_a)$

or $$E = K(e_s - e_a) \qquad ...(7.40)$$

where E = Evaporation rate in mm per day.

e_s = Saturation vapour pressure in the thin air film adjacent to water surface at corresponding temperature of water, called the *saturation vapour pressure of water.*

Values of e_s at different temperatures are given in Table 6.1.

e_a = The actual vapour pressure of the air above.

K = A constant, whose value depends upon various factors, such as, barometric pressure, wind velocity, humidity, etc.

It is evident from the above equation that *evaporation will continue only so long as* $e_s > e_a$. *However, no evaporation will occur when vapour pressure in the air above, becomes equal to the saturation pressure, thereby leaving no saturation deficit.*

7.34.2.2. Influence of depth on the rate of evaporation. For a large water surface, the evaporation rate is vastly influenced by the depth of water. To explain this, let us consider two water surfaces. One having a smaller depth of water, say for example, a shallow lake; and the other having a greater depth of water, say for example, a deep lake.

The rate of evaporation is different from these two surfaces, mainly because the entire body of the shallow lake will attain the air temperature easily, but the entire body of a deep lake will not easily attain the air temperature.

At the start of the summer season, the temperature of the entire body of water of the shallow lake will rise, and hence normal evaporation will take place. But in a deep lake, the temperature of water will remain much less than that of the air above, which will reduce the evaporation from deep lakes, because of the following two reasons:

1. When the temperature of water is less, the saturation vapour pressure is less, and hence lesser evaporation will occur (Dalton's law).

2. When the temperature of water is less than that of the air above, the film of air in contact with water is colder and, therefore, heavier and more stable, which nullifies the effect of wind. The vapour will, therefore, find more difficulty in dispersing, and hence reducing evaporation.

It is evident from the above discussion that the *increase of depth of water reduces the rate of evaporation from the surface of the water body in summer season.*

As the winter starts, the temperature of the surrounding air falls, but the temperature of water in the deep lake does not fall so easily, and thus the water remains warmer than what it would be in a shallow lake. If the temperature of water is more, the evaporation will be more. Secondly, the temperature of thin film in contact with water surface, is more than that of the air above. The contact film is thus warmer, lighter and heavier and, therefore, easy to disperse; causing more evaporation. *It follows that the increase of depth increases the rate of evaporation in winter season.*

7.34.2.3. Factors effecting evaporation losses. The evaporation losses from a water surface depend upon the following factors :

1. Area of the water surface ;
2. Depth of the water in the water body ;
3. Humidity ;
4. Wind velocity ;
5. Temperature;

6. Atmospheric pressure; and
7. Quality of water.

These factors are briefly described below :

(1) **Area of the water surface.** The amount of evaporation is directly proportional to the area of evaporation. If the exposed area is large, the evaporation will be more, and *vice-versa.*

(2) **Depth of water in the water body.** The depth of water influences the evaporation considerably. More depth reduces the summer evaporation, and increases the winter evaporation, as discussed in the previous article.

(3) **Humidity.** If the humidity of the atmosphere* is more, the evaporation will be less. Because during the process of evaporation, water vapour move from the point of higher moisture content to the point of lower moisture content, and the rate of this movement is governed by the difference of their moisture contents or the **moisture gradient** existing in the air. So, if the humidity, as measured by a hygrometer, is more, the evaporation will be less.

(4) **Wind velocity.** The process of evaporation also depends upon the prevailing turbulence in the air. If the turbulence is more or in other words if the velocity of the air in contact with water surface is more, the saturated film of air containing the water vapour will move easily, and the diffusion and dispersion of vapour will become easier, causing more evaporation, and hence more evaporation losses.

(5) **Temperature.** The process of evaporation also depends upon the temperature. If the temperature is more, the saturation vapour pressure increases, and the evaporation increases. Thus, in summer season or in hot countries, the evaporation will be more as compared to that in the winter season or in cold countries.

(6) **Atmospheric pressure.** If the atmospheric pressure (e_a) is more, naturally there will be lesser evaporation (Dalton's law). At higher altitudes, the atmospheric pressure is less and hence the evaporation should normally be higher. However, this is not exactly so, because of the decrease of temperature at higher altitudes, which reduces the evaporation.

(7) **Quality of water.** The quality of water in the water body also affects the rate of evaporation, since the presence of any dissolved salts in water reduces the saturated vapour pressure of water (e_s), which consequently reduces the rate of evaporation. For this reason, usually, the evaporation decreases by 1% for every 1% increase in the salinity of a water body. Turbidity may also affect the rate of evaporation by affecting the heat transfer within the depth of the water body.

7.34.2.4. Measurement of Evaporation. The rate of evaporation from large water surfaces, such as rivers, ponds, reservoirs, etc., can be determined by the following four methods:

1. Pan measurement method;
2. Using Empirical formulae;
3. Storage equation method; and
4. Energy budget method.

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*Relative humidity (RH) $= \frac{e_a}{e_s}$, where e_a = the atmospheric pressure, and
e_s = the saturation vapour pressure of water

These methods are described below in details:

(1) **Pan-measurement method for measuring evaporation losses.** In this method, a pan of certain standard dimensions is taken, and the water is filled in this pan up to a certain range of level. The depth of water evaporated from this pan of given area and in a given time is measured with the help of a micrometer ; from which the rate of evaporation per unit time per unit area is calculated. The rate of evaporation so obtained is multiplied by a suitable pan coefficient, so as to obtain the evaporation rate from the given water body. *This is necessary because it is found that the evaporation from a large surface source is not the same as that from a small pan.* Different shapes of pans have been designed by different designers, and different values of pan coefficients have been given. The more important of these pans are discussed below :

(*a*) *U.S., Weather Bureau Class A Pan.* This pan is 1.2 m in diameter, 25 cm deep, and its bottom is raised 15 cm above the ground surface, as shown in Fig. 7.61. The depth of water is to be kept in a fixed range, such that the water surface is at least 5 cm and never more than 7.5 cm below the top of the pan. Pan coefficient is about 0.6 to 0.8, generally taken as 0.7.

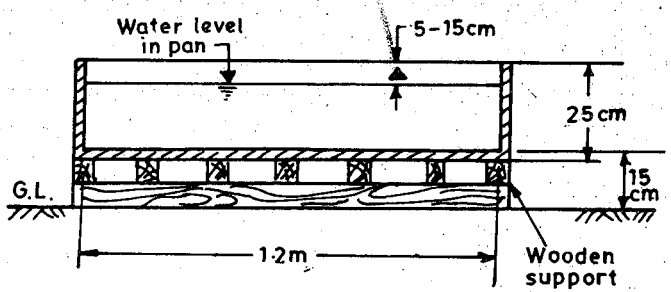


Fig. 7.61. U.S. Class A plan.

(*b*) *Colorado Sunken Pan.* It is 90 cm square with a depth ranging between 30 to 90 cm and buried in the ground within about 10 cm of the top. The water should be within 2.5 cm of ground level, as shown in Fig. 7.62. Pan coefficient is 0.75 to 0.86, generally taken as 0.78.

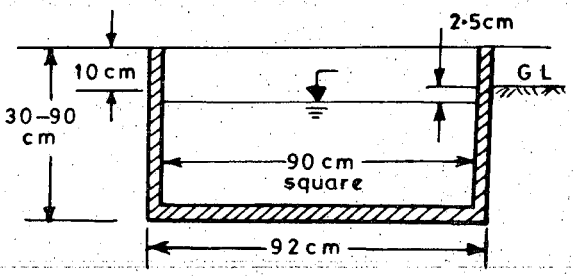


Fig. 7.62. Colorado Sunken Plan.

(*c*) *U.S. Geological Survey Floating Pan.* This is 90 cm (3′) square, 45 cm (18″) deep pan floating in a large body of water supported by drum floats in the centre of a raft 14′ by 16′. The water level in the pan is to be at the same level as that of the surrounding body of the water, with the sides of the pan projecting 7.5 cm (3″) above. The pan coefficient is generally taken as 0.80.

(*d*) *I.S.I. Standard Pan.* This pan evaporimeter, as specified by *IS-5973-1970*, is also known as *modified class A pan*. Such a pan is shown in Fig. 7.63. The evaporation from this pan is found to be approximately 14% less than that of Class *A* pan, which is an unscreened pan.

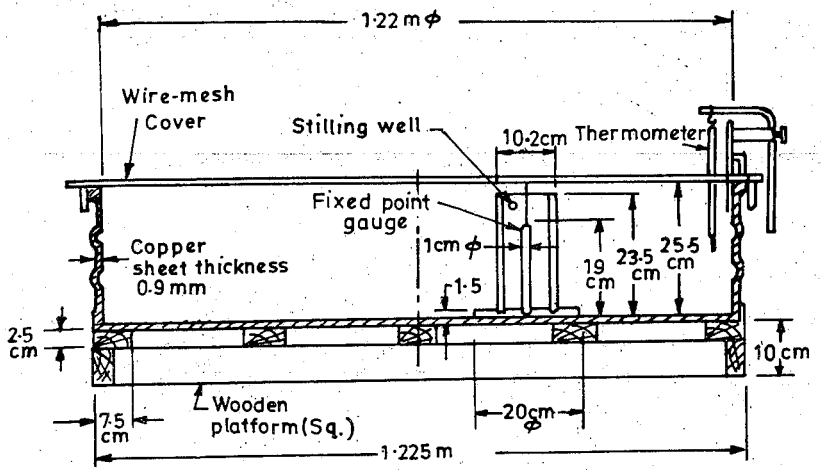


Fig. 7.63. ISI Evaporation pan.

This pan is infact, similar to class *A* pan, with 1.225 m dia, and 25.5 cm depth; but is **covered** at the top by a *hexagonal wire metting* of galvanised iron to protect the water in the pan from birds, and also to make the water temperature more uniform during day and night. The pan is placed over a square wooden platform of 1.225 m width and 10 cm height, to enable circulation of air underneath the pan. The pan coefficient is around 0.8.

(2) **Use of empirical formulae for estimating evaporation losses.** Various empirical formulas have been suggested for estimating the rate of evaporation per day from a large surface of water. These formulas are based on Dalton's law of evaporation. The two formulas which are generally used are given below:

(*a*) **Meyer's formula.** It states that :

$$E = K_m (e_s - e_a)\left(1 + \frac{V_9}{16}\right) \qquad ...(7.41)$$

where E = evaporation from the water body in mm/day

e_s = saturation vapour pressure at the water surface temperature in mm of mercury

e_a = actual vapour pressure of overlying air in mm of mercury at specified height (9 m here).

V_9 = monthly mean wind velocity in km/h at about 9 m above the ground

K_m = a coefficient, having value of 0.36 for large deep waters, and 0.50 for small and shallow waters.

(*b*) **Rohwer's formula.** It states that

$$E = 0.771\,(1.465 - 0.000732\,P_a)\,(0.44 + 0.0733\,V_{0.6})\,(e_s - e_a) \quad ...(7.42)$$

where E, e_s and e_a have the same meaning as in Eqn. (7.41)

P_a = mean atmospheric pressure *i.e.* barometric reading in mm of mercury

$V_{0.6}$ = mean wind velocity in km/h at ground level, which can be considered at 0.6 m above the ground.

The above formulas can be easily applied by meteorologically observing e_a and P_a. The wind velocity (V) will usually be measured at some other height above the ground than what is required above; which value can be used to compute V_9 or $V_{0.6}$, using

$$V_z = C \cdot Z^{\frac{1}{7}} = 0.143 \quad ...(7.43)$$

where V_z = velocity at any height Z

$$\therefore \quad \frac{V_{z_1}}{V_{z_2}} = \left(\frac{Z_1}{Z_2}\right)^{0.143} \quad ...(7.43a)$$

Example 7.15. *A reservoir with a surface area of 300 hectares has the following average meteorological values during a given week.*

water temperature	$= 30°C$
relative humidity	$= 50\%$
wind velocity at 1 m above ground	$= 12$ *km/h*
Mean barometer reading	$= 750$ *mm of Hg.*

Estimate the average daily evaporation from the lake reservoir, and the volume of water evaporated from the lake during this week. Make use of Meyer's formula as well as Rohwer's formula to compare the results.

Solution. (*a*) **Using Meyer's formula,** we have eqn. (7.41) as:

$$E = K_m\,(e_s - e_a)\left(1 + \frac{V_9}{16}\right)$$

where $K_m = 0.36$ for large deep water

e_s = saturation vapour pressure at 30°C

= 31.82 mm of Hg (from Std. table 7.23)

e_a = ?

Also, $\frac{e_a}{e_s} = 50\%$ (R.H.)

$\therefore \quad e_a = 50\%\ e_s = 0.5 \times 31.82$

$= 15.91$ mm of Hg

V_9 = ?

$V_1 = 12$ km/h

$$\frac{V_9}{V_1} = \left(\frac{9}{1}\right)^{0.143}$$

$$\therefore \quad V_9 = (9)^{0.143} \times 12 \text{ km/h} = 16.43 \text{ km/h}$$

Substituting values, we get

$$E = 0.36\,(31.82 - 15.91)\left(1 + \frac{16.43}{16}\right) = \mathbf{11.61\ mm/day.} \quad \textbf{Ans.}$$

$\therefore$ Total evaporation volume in 7 days (1 week) from 300 hectares of surface area:

$$= \frac{11.61}{1000} \times 7 \times (300 \times 10^4)\ \text{m}^3 = 2{,}23{,}810\ \text{m}^3. \quad \textbf{Ans.}$$

(*b*) **Using Rowher's formula**, we have eqn. (7.42) as :

$$E = 0.771\,(1.465 - 0.000732\,P_a)\,(0.44 + 0.0733\,V_{0.6})\,(e_s - e_a)$$

where $e_s = 31.82$ mm of Hg

$e_a = 15.91$ mm of Hg

$P_a = 750$ mm of Hg

$$\frac{V_{0.6}}{V_1} = \left(\frac{0.6}{1}\right)^{0.143}$$

or $$V_{0.6} = \left(\frac{0.6}{1}\right)^{0.143} \times 12\ \text{km/h} = 11.15\ \text{km/h}$$

$$E = 0.771\,(1.465 - 0.000732 \times 750)\,(0.44 + 0.0733 \times 11.15) \times (31.82 - 15.91)\ \text{mm/day}$$

$$= \mathbf{14.13\ mm/day} \quad \textbf{Ans.}$$

$$\text{Total evaporation} = \frac{14.13}{1000}\,\frac{m}{\text{day}} \times 7\ \text{days} \times (300 \times 10^4\ \text{m}^2) = \mathbf{2{,}96{,}730\ m^3}. \quad \textbf{Ans.}$$

Conclusion. The evaporation loss as computed by Rohwer's formula is about 20% higher than that calculated by Meyer's formula. *Hence, such formulas give only rough approximate values. However,* Rohwer's formula is generally preferred to all others.

(3) **Storage equation or Water-budget method for estimating evaporation losses.** The third method to determine the evaporation losses from a reservoir or a river, etc., is by the use of storage equation, which is given by the following expression:

$$P + Q_i \pm Q_u = E + Q_o \pm \Delta Q_s \qquad \ldots(7.44)$$

where P = Total precipitation on the water surface.

Q_i = Total surface inflow

Q_u = Total underground inflow or outflow (+ve value for inflow, and –ve value for outflow).

E = Evaporation from the water surface

Q_o = Surface outflow

ΔQ_s = Change in storage (+ve value for an increase in storage, and –ve value for a decrease)

All these factors are converted into the same units, generally expressed as the depth on the water area for some convenient time interval. If all other terms are known, E can

be calculated. The method is not very useful, because the various quantities cannot be easily worked out.

(4) **Energy budget method to estimate evaporation losses.** This method is based on the principle of conservation of heat energy and the cooling produced by evaporation. Thus, the energy available for evaporation is determined by considering the incoming energy, outgoing energy, and energy stored in the water body, over a known time interval.

Let us consider a water body, as shown in Fig 7.64. The energy balance at the evaporating surface, in a period of one day (say), is then given as:

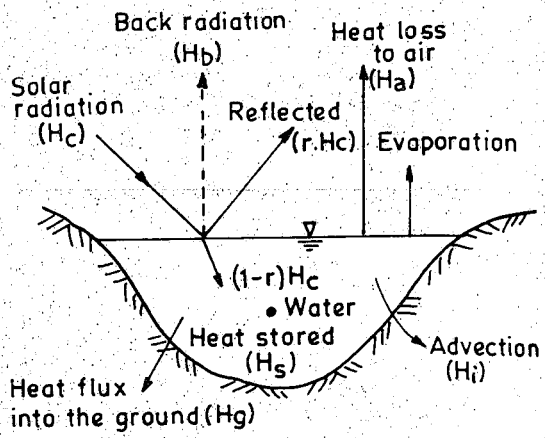


Fig. 7.64. Energy balance in a water body.

(*i*) Net heat energy received or *Net incoming energy*

$$H_n = H_c(1-r) - H_b \qquad ...(7.45)$$

where $H_c(1-r)$ = the incoming solar radiation into a surface of *reflection coefficient* (**albedo**) r, as shown in Fig. 7.64. Value of r for water surface approximates 0.05 ; and for newly laid snow = 0.90

H_b = back radiation (long wave) from water body.

(*ii*) *Outgoing energy* $= H_a + H_g + H_i$

where H_a = sensible heat lost to air

H_g = heat lost to ground

H_i = net heat conducted out of the system by water flow (*advected energy*).

(*iii*) *Heat stored in the water body* (H_s).

(*iv*) *Heat used in evaporation* $(H_e) = \rho_w \cdot L_a \cdot E$

where ρ_w = Density of water

L_a = Latent heat of vaporisation of water

E = Evaporation in mm.

Using energy conservation law, we have :

Total incoming energy = out going energy + energy stored + energy used in evaporation

or $$H_n = (H_a + H_g + H_i) + (H_s) + (H_e) \qquad ...(7.46)$$

All the energy terms in the above eqn. are in calories/sq mm/ day.

Moreover, all the terms except H_a in eqn. (7.46) can be measured or evaluated indirectly. The sensible heat transfer to air (H_a) can, however, be estimated by using **Bowen's ratio** (β), given by the equation:

$$\beta = \frac{\text{Sensible heat transferred by conduction or convecton from the water surface to the atmosphere } (H_a)}{\text{Energy used in evaporation } (H_e)}$$

$$= 6.1 \times 10^{-4} P_a \left(\frac{T_w - T_a}{e_s - e_a} \right) \qquad ...(7.47)$$

where P_a = Atmospheric pressure in mm of Hg

T_w = Temperature of water in °C

T_a = Temperature of air in °C

e_s = Saturation vapour pressure at T_w °C, in mm of Hg

e_a = Vapour pressure of air in mm of Hg.

Eqn. (7.46) and (7.47) can further be solved as :

$$H_n - H_g - H_i - H_s = H_a + H_e \qquad ...(i)$$

and $$\frac{H_a}{H_e} = \beta$$

or $$1 + \beta = 1 + \frac{H_a}{H_e} = \frac{H_a + H_e}{H_e}$$

or $$H_a + H_e = (1 + \beta) \cdot H_e$$

$$= (1 + \beta) \cdot \rho_w \cdot L_a \cdot E \qquad ...(ii)$$

Substituting this value in eqn. (*i*), we get

$$H_n - H_g - H_i - H_s = (1 + \beta)\, \rho_w \cdot L_a \cdot E \qquad ...(iii)$$

or $$\boxed{E = \frac{H_n - H_g - H_i - H_s}{\rho_w \cdot L_a (1 + \beta)}} \qquad ...(7.48)$$

E can thus, be computed by measuring all the above terms of H, and calculating β by eqn. (7.47). For short periods of less than a week, H_i and H_s can be neglected as negligibly small; and eqn. (7.48), then reduces to

$$E = \frac{H_n - H_g}{\rho_w \cdot L_a (1 + \beta)} \text{ (for small periods)} \qquad ...(7.49)$$

Estimation of evaporation from a lake by the above method gives satisfactory results, with errors of the order of 5% when applied to periods less than a week.

There are certain other methods, like *mass-transfer method,* etc., to determine evaporation losses; but the most important and widely used is the first one, *i.e.*, the pan measurement method.

Example 7.16. *At the beginning of a certain week, the depth of water in an evaporation pan, 1.2 metres diametre, was 7.75 cm. During the week, the rainfall was 3.8 cm, and 2.5 cm of water was removed from the pan to keep the depth of water in it within a fixed range. At the end of the week, the gauge indicated a depth of 8.32 cm of*

water in the pan. Using a suitable evaporation pan coefficient, estimate the evaporation during the week from the surface of a reservoir under similar atmospheric conditions.

(U.P.S.C., Engg. Services, 1970)

Solution.

Initial depth of water in the pan	= 7.75 cm
Rainfall	= 3.8 cm
Water removed	= 2.5 cm
Net addition of water in the pan	= 3.8 – 2.5 = 1.3 cm
Depth at the end of week, if there is no loss	= 7.75 + 1.3 = 9.05 cm
Actual depth at the end of week	= 8.32 cm
Evaporation lost from the pan	= 9.05 – 8.32 = 0.73 cm
Evaporation loss from the reservoir	= 0.7 × 0.73 = 0.511 cm [Assume a pan coefficient = 0.7]
Hence, the evaporation during the week from the surface of reservoir under similar atmospheric conditions	**= 0.511 cm. Ans.**

7.34.2.5. Conservation of reservoir water. In hot and humid regions, particularly in arid and semi arid region, water is available with a great difficulty. Out of the already scarcity available water, considerable amount is further lost in evaporation. The water sceruly may become very acute in desert areas, where availability of water is already very small, and evaporation is higher due to hot weather. In all such cases, water must be protected by minimising the evaporation losses.

A striking example of a severe reservoir evaporation loss is offered by the Steven's Creek Reservoir in Australia; the annual precipitation in this region is less than 25 cm, and it was reported that about 3 litres of water was evaporated for every 1 litre of water pumped. So in all such cases, the water losses must be reduced to increase the water supply.

In order to reduce these evaporation losses from a reservoir, the most adopted method is to sprinkle a layer of some chemical compound over the surface of water, which may reduce the activity of evaporation. Many chemical compounds have been developed, but the most important and commonly used is **hexadecanol;** chemically known as **Cetyl alcohol.** The advantages of this compound are that it is cheaper and does not impart any colour, odour, or biological activity to the water. However, it forms a thin film on the surface of water, which is very fragile, and may break even by blow of winds. The issue of reservoir evaporation losses and methods adopted for their reduction are discussed in more details in article 18.17.1.

7.34.3. Transpiration Losses. Transpiration is the process by which the water leaves a living plant, through its leaves, to enter the atmosphere as water vapour. Large amount of water, much-much more than what is really consumed by the plant, is thus, given away by the plant to the atmosphere. *The ratio of water (moisture) thus transpired by a plant to the atmosphere, to the actual water consumed in its nourishment, may be*

as high as 800 or even more. The moisture consumption in actual nourishment for building plant tissues, during its metabolic process, is thus, negligibly small, and hence, generally ignored. Heavy quantities of water from the surface of the Earth are, thus, lost by trees, plants, and other vegetation, through transpiration; and hence, *transpiration constitutes an important part of the hydrologic cycle.*

Transpiration, infact, occurs, when the plant manufactures carbohydrates for its growth, by the process of **photosynthesis.** What actually happens is, that the water extracted by the plant-root system from the soil, is transferred to the plant leaves; and on the surface of each plant leaf, photosynthesis occurs, where the air enters the *stomata* openings in the leaf surface, and the *chloroplasts* within the leaf utilise carbon dioxide from the air along with a very very small portion of the available moisture, to manufacture carbohydrates. *As air enters the leaf, the water escapes through the stomata.*

In addition to this escape of water to the atmosphere (*i.e.* transpiration), water is also lost during the growth of the plant by evaporation from the soil body* surrounding the plant. This loss of water may strictly be called as evaporation. Since in the process of vegetation growth, it is generally not feasible to separate the transpiration and its connected evaporation from the plant's surroundings, a *combined measure* is obtained, and this is called **evapo-transpiration** (ET).

It should be understood that transpiration occurs during photosynthesis, which usually occurs during the sun-light; and hence, most of the transpiration will occur during the day light hours. So much so that about 95% of the total transpiration occurs during the day time alone. On the other hand, the evaporation continues throughout the day and night, although its rate may be different. Moreover, the transpiration loss, not only depends upon the available sun light, but also upon the available moisture, since plants transpire less moisture when its availability becomes scarce. It will also be influenced by the type and characteristics of the plants. Different plants will evidently transpire different amount of water; and, hence, their water consuming characteristics are compared by a term, called *transpiration ratio.*

7.34.3.1. Transpiration ratio (T.R.) is defined as the ratio of the mass of water transpired by the plant in its full growth period, to the mass of the dry matter produced.

Thus,

$$\text{T.R.} = \frac{\text{Total mass of water transpired by the plant during its full growth}}{\text{mass of dry matter produced}} \quad ...(7.50)$$

Evidently, the TR has no Units.

"Mass of dry matter produced," is generally taken as the entire mass of the plant including its roots; but sometimes, only the marketed or the useful crop, such as grain of wheat, gram, etc. is weighed.

For a given kind of plant, the T.R. varies widely, depending upon the region and its climate. T.R. of a crop will increase if the water requirement of the crop is more. That is why the T.R. of wheat is of the order of 300–600 (average = 450) and that of the rice is of the order of 600–800 (average = 700).

* This also includes the water body, when plants grow within a water body.

7.34.3.2. Transpiration measurement. In order to measure the transpiration, various methods are used by botonists. Phytometer method, which is generally adopted by engineers, is, however, discussed here.

Phytometer method. This is the most important and widely used method. The instrument consists of a closed and water-tight tank, containing sufficient earth to nourish the plant. It is sealed, so as to prevent any automatic inflow or outflow of water. Water is applied, only artificially, till the plant growth is completed. The instrument is weighed in the beginning as well as at the end of the experiment. Water required during the growth is measured, and the loss due to transpiration can be calculated by the equation :

$$T = (W_1 + W) - W_2 \qquad ...(7.51)$$

where W_1 = Initial weight of the instrument

W_2 = Final weight of the instrument

W = Total weight of water added during full growth of the plant.

T = Loss due to transpiration.

But one must not forget that the transpirational loss calculated above, is not the actual value of the field, but is only of the lab. Some influence is produced by the protected growth and due to the difference left in exactly reproducing the field soil conditions in the lab. So the values obtained by the above test must be multiplied by a suitable factor to obtain the possible field results. The effect of the shape of phytometer upon the water requirement of the plant is still not known. The growth in screened shelters has, however, been found to reduce the water requirement by about 10% or so.

7.34.3.3. Evapo-transpiration (ET). As defined above, the evapo- transpiration is the sum of the water lost to the atmosphere by the plants through transpiration, and the water evaporated from the soil or water body, surrounding the plant. This loss will evidently depend upon the available moisture for both the processes of transpiration and evaporation. When sufficient moisture is freely available to completely meet the needs of the vegetation fully covering an area, the resulting evapo-transpiration is called **potential evapo-transpiration** (PET). The real evapo-transpiration, occurring in a specific situation in the field, is called the *actual evapo transpiration* (AET).

It has further been established that *whereas, PET critically depends upon the climatological factors, rather than on the characteristics of plants and soils ; the AET is largely affected by the characteristics of soil and vegetation.*

Actual evapo-transpiration (AET) in the field can be measured by an instrument called a *lysimeter*. A **lysimeter**, like a phytometer, is a special water-tight tank, containing a block of soil, and is installed in a field of growing plants. Same plants are grown in the lysimeter, as in the surrounding field. The amount of water required to maintain constant moisture condition within the tank are measured either volumetrically or gravimetrically through an arrangement made in the lysimeter. This value of water required, when averaged over the crop period, in mm/day, will give us the value of the actual evapo-transpiration (AET) in mm/day for that crop/vegetation, under these specific field conditions.

A lysimeter should be so designed, as to accurately reproduce the field conditions of soil, moisture content, and type and size of the vegetation of the surrounding area. The instrument should also be buried in the field, so that the soil is at the same level inside, as well as outside the container. Such field studies are time consuming and costlier, and are generally adopted only for research studies on crops.

7.34.3.4. Consumptive use (C_u). Consumptive use may be defined as the amount of water used in the evapotranspiration from an area under vegetation, plus the water used by the plants in their metabolic process for building of plant tissues. Since the quantity of water used in metabolic process is insignificant compared to evapotranspiration, *the terms 'consumptive use (C_u)' and 'evapotranspiration (E_t)' are considered as one and the same thing.*

Several equations have been suggested to estimate the consumptive use of different crops, or for different segments of a basin, depending upon the type of vegetation covering each sub basin.

The equations, which are generally used for estimating consumptive uses of different crops, *i.e*, (*i*) *Ballaney Criddle Equation*, and (*ii*) *Hergreaves class A Pan Evaporation value* (whether determined in the field or computed by *Christiansen equation*), have already been described in details in article 2.13, along with *Penman's equation,* which is used these days to determine the *consumptive use values of different areas.*

7.34.4. Land Evaporation Losses. Land evaporation is the loss of moisture due to the direct evaporation from the individual soil grains, located at or near the ground surface. This evaporation is governed by all those factors which govern the evaporation from water surfaces, and additionally on the *presence of moisture in the soil.* Since the moisture in the soil will vary throughout the year, the land evaporation will also vary throughout the year. In order to correlate the evaporation from a land surface with the evaporation from a water surface, a term called *evaporation opportunity* is usually adopted, as defined below :

7.34.4.1. Evaporation opportunity (E.O.) It is the percentage which the actual land evaporation bears to the evaporation from an equivalent free water surface. The evaporation from the free water surface is known as *evaporation.* Thus, evaporation opportunity can be expressed as

$$\text{E.O.} = \frac{\text{Actual evaporation from the land at a given time}}{\text{Evaporation from an equivalent free water surface}} \times 100 \qquad ...(7.56)$$

The land evaporation depends upon the watertable. If water-table is high, the land evaporation will be more; and if it is lowered, land evaporation will reduce. It also follows that if the ground surface is within the capillary fringe (capillary fringe extended), the evaporation opportunity will be almost uniform, and if the ground surface is never within the capillary fringe, the rate of land evaporation will vary throughout the year.

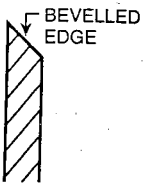
STREAMFLOW MEASUREMENT AND GAUGE DISCHARGE CURVES

Streamflow as we all know, is *the discharge that is found flowing in a stream channel at a given time, and at a given section (i.e., location)*. It thus, includes surface runoff as well as ground water flow-that has seeped into the stream.

The process, by which the discharge of a stream channel is measured, is known as *stream-gauging*. The word **stream-gauging,** thus, means, gauging the stream, *i.e.* to determine the characteristics of streamflow, and includes determining the river discharge over long periods of time, and determining the velocity of flow, if needed.

7.35. Installations used for Measuring Discharges in Open Channels and Rivers

Several types of hydraulic structures, like: *notches, weirs, flumes,* and *drops* are used for measurement of flow in open channels, besides *velocity-area method, power plant method, dilution method, slope-area method,* and certain other methods, as discussed below :



7.35.1. Notches for Measuring Discharge. Notches or **gauge plates** are cut from thick metal plates, and may be *triangular* or *trapezoidal* with shampher/bevelled edge. They are largely used for measuring discharges in laboratories, and at very small flows in the field.

(1) **Triangular (V) Notches.** 90° triangular notches are used for measuring small quantities of flow in open channels, up to about 1.25 cumecs.

The approach channel for fixing this installation, should be reasonably smooth, free from disturbances, and straight for a length equal to atleast 10 times the width. The structure, in which the notch is fixed, should be rigid and watertight, with its vertical upstream face. The downstream level should be always atleast 15 cm below the bottom most portion of the notch,* ensuring free flow.

The head causing flow over the notch is measured by *standard hood gauge,* upstream at a distance of 3 to 4 times the maximum depth of flow over the notch.

The discharge Q for the V-notch is then, calculated by the equation:

$$Q = \frac{8}{15} C_d \cdot \sqrt{2g} \cdot \tan \frac{\theta}{2} \cdot H^{5/2} \qquad ...(7.57)$$

where H = Head over the notch sill

θ = angle of the notch at the centre

g = Acceleration due to gravity

= 9.81 m/sec^2

C_d = Effective discharge co-efficient, the values of which may vary from 0.6 to 0.69 for values of head varying from 0.05 to 0.4 m, or so.

* Inverted apex.

The triangular notches should be used only when the head is more than 0.06 m (*i.e.* 6 cm). The method provides quite accurate values of discharge, and the obtained discharge values may vary from 97% to 103% of the true discharge values for discharges varying from 0.008 to 1.25 cumecs.

For 90° triangular notch, $\theta = 90°$, the eqn. (7.57), becomes:

$$Q = \frac{8}{15} C_d \cdot \sqrt{2g} \cdot \tan 45° \, H^{5/2}$$

or $$Q = 2.36 \, C_d \cdot H^{5/2} \text{ (for 90° V-notch)} \quad ...[7.57\ (a)]$$

(2) **Rectangular Notches.** Rectangular notches having atleast 15 cm width, may sometimes be installed, instead of using a *V*-notch. Such a rectangular notch with end contractions, is governed by the discharge formula:

$$Q = \frac{2}{3} C_d \cdot \sqrt{2g} \cdot b_e H_e^{3/2} \quad ...(7.58)$$

(with end contractions)

where b_e = effective width or length of the notch, as against its actual width b, where $b_e = b + k$

where k varies from 2.5 mm, 3 mm, and 4 mm for b/B values of upto 0.4, 0.4 to 0.6, and 0.6 to 0.8, respectively. B being the width of channel

H_e = effective head over the notch

= actual head measured (H) + 1 mm

C_d = coefficient of discharge, varying from 0.58 to 0.70 for values of $\frac{b}{B}$ from 0 to 0.8, where B is the width of the channel.

For a rectangular notch, without end contractions, the discharge formula is given by:

$$Q = \frac{2}{3} C_d \cdot \sqrt{2g} \cdot b \cdot (H_e')^{3/2} \quad ...(7.59)$$

(without end contractions)

where b = width or length of the notch.

H_e' = effective head

= actual measured head (H) + 1.2 mm

$$C_d = 0.602 + 0.075 \frac{H}{p}$$

where p = height of the bottom of the notch from the bed of the channel.

H = head over the notch sill

(3) Trapezoidal Notches (Cipoletti Notches). The discharge formula for a trapezoidal notch is given by:

$$Q = 1.859 \cdot b \cdot H^{3/2} \qquad ...(7.60)$$

where b = bottom width or length of the notch
H = head over the notch sill.

The main advantage of such a notch is that: *as the flow passes over the notch, the end contractions either get eliminated or considerably reduced.*

The sides of the notch should have a slope of 1 H : 4 V, so that the top width of discharge equals to the bottom width of the notch (b) + $\frac{1}{2}$ the head of water over the sill of the notch (H).

Concrete or Masonry Weirs for Measuring Discharges. Weirs are similar to rectangular notches, but having considerable thickness in the direction of flow (t), and, thus, their coefficient of discharge is less than that of a notch.

(a) The discharge equation for a **suppressed weir*** (weir without end contractions) is given by the eqn.:

$$Q = C_d \cdot \sqrt{2g} \cdot L \cdot \sqrt{Hh^2 - h^3} \qquad ...(7.61)$$

where L = the width or length of the weir, across the flow (equivalent to b of notches).
h = dropped measured head of water over the middle of the weir.

The condition for maximum discharge for such a **broad crested weir** is:

$$h = \frac{2}{3} H; \text{ and hence}$$

$$\boxed{Q_{max} = 1.7\, C_d \cdot L \cdot H^{3/2}} \qquad ...(7.62)$$

where L = width of the weir flow.
H = upstream water head above the weir crest
C_d = 0.864 to 1.0, depending upon $\frac{H}{t}$ ratio (*i.e.* ratio of head to the thickness of weir in the direction of flow), varying between 0.4 to 1.6. For $\frac{H}{t}$ values lower than 0.4, C_d may again be taken as equal to 0.864.

(b) For a *weir with end contractions*, the term L as used above in equation (7.62), may be replaced by L_e *i.e.* effective width of flow,

where $$L_e = L - 0.1\, n \cdot H \qquad ...(7.63)$$

where n = No. of end contractions.

The weirs should be used for measuring discharges in open channels only when the head is more than 0.06 m, and the minimum length of weir (L) should be 0.3 m.

The discharge values obtained by weir measurements may vary from 95 to 105% of the true discharges.

7.35.2. Control Meters or Standing Wave Flumes for Discharge Measurements in Streams. A **control meter** is a structure, constructed across a stream, by which *critical depth** can be produced by changing a subcritical* channel flow into a supercriticall* flow, and *vice versa.* Such an arrangement, thus involves the formation of a *standing wave* or a *hydraulic jump;* and hence, also called a **standing wave flume.**

The above purpose can be achieved either by *raising the bottom of the channel* (Fig. 7.66), or by *fluming the width of the channel* (Fig. 7.67).

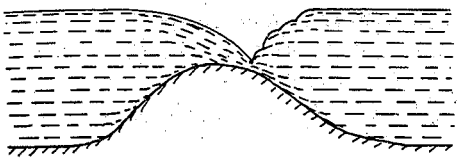


Fig. 7.66. Raised crest along channel.

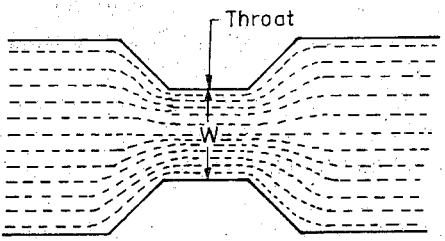


Fig. 7.67. Constricted width along channel.

The throat of the section is made rectangular or trapezoidal. The floor of the throat is almost level, whereas the floor of the expanding outlet is given a steep slope, sufficient to cause the water to leave the throat at supercritical velocity, and thus to ensure the presence of critical depth at some point in the throat. Hence a control meter, if properly designed, will be associated with the phenomenon of hydraulic jump.

If the original depth (y_1) in the channel is more than the critical depth (y_c), then the jump will be formed below the constriction; but if y_1 is less than y_c, then the jump will be formed above the control.

It is not possible to measure y_c because the exact position of its occurrence in the throat varies with the discharge, and is not easily found even for a given discharge. The discharge can be determined by applying the relationship between discharge Q and sp. energy at critical depth, as below:

* The field test for assessing the state of flow in a river to know as to whether it is super-eritical, sub-critical, or critical, we can drop a small piece of wood in the flow and observe the movement of ripples so formed on the surface of water. If the ripples move downstream with current, then the flow is super-critical if the ripples move upstream in opposition to the flow current, the flow is sub critical ; and if ripples remain steady, the flow will be critical (*i.e.* in standing wave condition).

Let E_c represents the minimum sp. energy at critical depth (pt. A) as shown in Fig. 7.68. Then applying Bernoulli's equation between pt. 1 and pt. A, we get

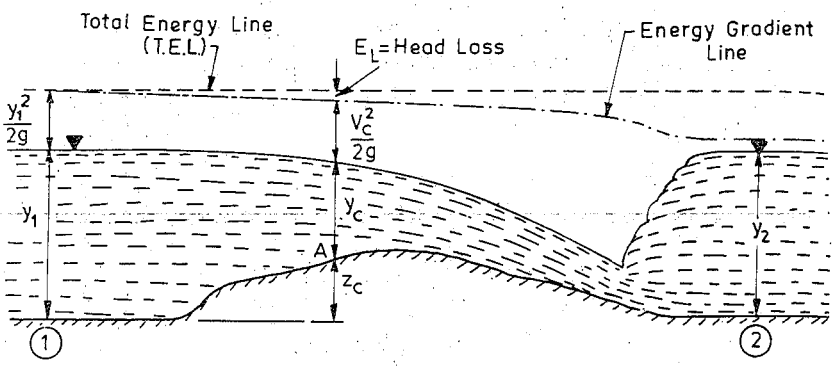


Fig. 7.68

$$y_1 + \frac{V_1^2}{2g} = \left[y_c + \frac{V_c^2}{2g} \right] + Z_c + E_L$$

where E_L = energy loss in the process of formation of hydraulic jump.

But $$y_c + \frac{V_c^2}{2g} = E_c = \text{Sp. energy at } A.$$

Therefore, $$y_1 + \frac{V_1^2}{2g} = E_c + Z_c + E_L$$

or $$E_c = \left[y_1 + \frac{V_1^2}{2g} \right] - Z_c - E_L \qquad ...(7.64)$$

The energy loss (E_L) is very small, especially when all the surfaces are connected by tangent curves. The length of the throat is kept as small as possible to avoid energy losses. But it must be long enough to make sure that the critical depth falls within the throat. A throat length of approximately three times the critical depth has been found to give satisfactory results. With the above assumption, the term E_L becomes small and can be ignored.

Therefore, equation (7.64) can be written as

$$E_c = \left[y_1 + \frac{V_1^2}{2g} \right] - Z_c \qquad ...(7.65)$$

y_1 and Z_c are known, V_1 can be found by trial, and hence, E_c can be computed.

Now, discharge Q is given by the relationship :

$$Q = C \cdot B_0 E_c^{3/2} \qquad ...(7.66)$$

$(\because Q \propto y_c^{3/2}$; and $y_c = \frac{2}{3} E_c)$

where B_0 = width of throat.

E_c = Sp. energy at critical depth, as shown.

C = a constant.

The value of E_c as computed from equation (7.65) can be used to evaluate Q by putting the values in equation (7.66).

Based upon the above principles, the final equation for discharge through a **constricted width type standing wave flume,** where ($Z_c = 0$), is given by :

$$Q = \frac{2}{3} C_f \cdot \sqrt{2g}\,(B_0 - mb - 2C_c\, mH)\, H^{3/2} \qquad ...(7.67)$$

where C_f = coefficient of friction, having the following values :

0.97 for $Q = 0.05$ to 0.3 cumec

0.98 for $Q = 0.3$ to 1.5 cumec

0.99 for $Q = 1.5$ to 15 cumec

1.00 for $Q = 15$ cumecs and above.

B_0 = overall throat width including piers.

m = No. of piers.

b = thickness of each pier.

C_c = coefficient of contraction having a value of 0.045 for piers with round noses; and 0.040 for piers with pointed noses.

$$H = y_1 + \frac{V_1^{2*}}{15.2}$$

where y_1 = depth upstream over sill of throat.

V_1 = mean vel. of approach.

Advantages of Standing Wave Flumes. *The control meters are preferred to weirs for larger flows and for silt-laiden rivers.* The greatest advantage of a control meter over a weir is its ability to measure discharge even in silt-laiden streams. The flow immediately above a weir is retarded, and therefore silt gets collected; while on the other hand, *in a control meter, the flow above the control gets accelerated and therefore silt and debris tend to be swept through the control,* and hence almost no silting takes place. Secondly, logs and debris damage a sharp-crested weir, while there is no such damage in a control-meter.

The capacity of a control-meter to measure discharge can be increased or decreased by making the throat wider or smaller respectively. But the lower limit increases with the capacity of the meter. Hence, a particular given control meter can not measure discharge lesser than its lower limit. Hence, to measure discharge below the lower limit

*Truly speaking the effect of velocity of approach is greater than $V_1^2/2g$, because the velocity in the central portion will be higher than the mean velocity V_1. Thus, the head due to velocity of approach is taken as $\frac{V_1^2}{15.2}$ instead of $V_1^2/2g$.

of large meters, a second smaller control meter or sometimes a weir is often installed in parallel with the larger one.

Limitations of Standing Wave Flumes. These flumes should be used only when the head is more than 6 cm; and ratio of $\frac{y_2}{y_1}$ (*i.e.*, downstream depth over the sill ÷ upstream depth over the sill of the throat) should always be greater than 0.5.

When the ratio y_2/y_1 is less than 0.5, then a *'drop'* may be used inplace of a *'flume'*.

The minimum width of the flume should be 9 cm.

The discharge values obtained by measurements with standing wave flumes would vary from 95% to 105% of the true discharges.

Parshall flume is also a type of constricted standing wave flume (Fig. 7.67), widely used for discharge measurements, which is given by:

$$Q = C \cdot W y_1^{2.58} \qquad ...(7.68)$$

where W = throat width in m.

y_1 = *U/s* gauged depth in m.

C = a constant, the value of which depends on throat size. The value of C is generally taken as 2.42 for almost entire range over which parshall flume is used, *i.e.* for Q varying from 0.001 cumec to 100 cumecs (i.e. throat widths varying from 7.5 cm to 15 m). This numerical factor is, however subject to even upto 40% variation in extreme cases (less in case of smaller widths).

The above eqn., therefore becomes

$$\boxed{Q = 2.42\, W \cdot y_1^{2.58}} \qquad ...(7.68\ a)$$

7.35.3. Venturi-Flumes or Venturi-meters for Measuring Discharges. A venturi-flume (ventri-meter), like a constricted standing wave flume, involves constricting the channel section at the throat, but the constriction does not produce critical conditions, involving change of flow from subcritical to supercritical, and *vice-versa.*

The discharge for such a **Venturi-meter**, is given by:

$$Q = 0.5445\, C_v \cdot C_e \cdot \sqrt{g}\, bH^{3/2} \qquad ...(7.69)$$

where C_v = the coefficient of velocity, varying from 1.04 to 1.15.

C_e = effective coefficient of discharge, varying from 0.885 to 0.99, depending upon H/l varying between 0.05 to 0.7, where l is the length of the throat in the direction of flow.

b = width of throat *perpendicular* to flow.

H = head of water in the throat section.

Venturi-flumes should be used only when head available is between 5 cm to 1.8 m, and minimum width of the flume should be 9 cm.

The discharge values obtained by measurements with venturi-flumes may vary from 95% to 105% of the true discharges.

7.35.4. Drops for Measuring Discharges. Canal falls along an open channel provide an easy method for roughly computing the flow discharge through the channel. The discharge formula in such a case, is given by:

$$Q = \sqrt{g} \cdot B \cdot y_c^{3/2} \qquad ...(7.70)$$

where B = width of the channel

y_c = critical depth

The above formula is well applicable, provided that there exists a minimum straight length of 20 times the end depth in the approach channel. For such cases, the ratio of the end depth to the critical depth has a value of 0.7, which has been accounted in the above formula.

This method is, however, limited to only those cases, where the width of the channel is atleast 0.3 m, and critical depth y_c is atleast 5 cm, which is generally the case in all canal drops.

The discharge values obtained by measurements at drops, may vary between 90% to 110% of the true discharges.

7.35.5. Velocity Area Method for Computing Streamflow. This is a direct method of computing the discharge in a stream by measuring its velocity and area of flow. Since the velocity may vary along the entire cross-section of the river, the entire stream cross-section is divided into a number of parts. The area of each part and the velocity of flow through it, is determined and multiplied, so as to get the various values of partial discharges. The summation of these partial discharges will give us the river-discharge.

The place where such measurements are taken is known as the **velocity area station**, and the method is known as the velocity area method.

The method, therefore, consists in stretching a cable across the river at the chosen velocity area station, and measuring distances across the river along the cable, and their corresponding water depths. The flow velocities in these divided segments are determined, generally by a *current meter* at different depths,* or approximately by *floats* or *velocity rods* at the surface, to compute segmental, and finally, the total discharge in the stream.

The flow velocity in a stream as pointed out above, can be measured by :

(1) *Surface and subsurface floats:*

(2) *Pressure instruments; and*

(3) *Current-meters, including Price's current meter.*

These three types of devices are explained below:

(1) **Surface and Sub-surface Floats.** A float, in general, is nothing but a piece of cork, apple, or anything else which can float on water. The distance travelled by such a float divided by the time taken to travel that distance, gives us the float velocity or the surface velocity of water. The mean velocity in the vertical can be calculated by

* To determine mean depth in the vertical, which is correctly obtained at 0.6 depth.

multiplying the surface velocity by a suitable coefficient (≈ 0.85) which is calculated by experiments.

Sometimes, wooden rods, painted with some water proofing paint are used as floats. A weight is tied to the bottom of such a rod, so as to keep it in water. The weight is so adjusted that the rod length equal to $0.94\,y$ remains inside water, where y is the depth of water. The top about 4 cm of the rod remains out of water, and remains visible. The rod is made to float for a distance ranging between 30 to 150 m or so, and the time taken by the rod to travel this distance is noted by a stop watch. The distance divided by time, will give us the velocity. These wooden rods are generally painted with some water proofing paint, so as to prevent them from being water-logged. These **velocity-rods** give the mean velocity directly.

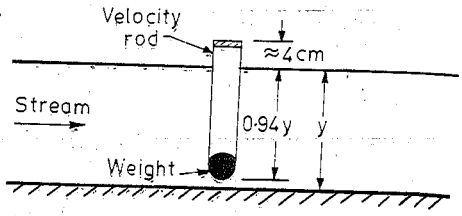


Fig. 7.69. Velocity rod.

The use of floats is generally restricted to straight rivers having almost uniform cross-section throughout. They are not of much use for natural streams, especially when the depth of water exceeds 1.5 m or so.

(2) **Pressure Instruments.** The velocity head of the moving water is converted into pressure head with the use of certain devices, called pressure devices. The pressure-head can be measured and the velocity calculated by equating their pressure head (in metres of water) to $\frac{V^2}{2g}$. **A pitot tube** is an example of this kind of instrument, but its use is restricted only to pipes or to experimental channels.

(3) **Current Meters.** A current meter is the best instrument for measuring the velocity of a natural stream, and is generally and universally adopted for this purpose. Various kinds of current meters are in use these days, but the most common and widely used in India is the *Price's Current Meter.*

Price's current meter is a differential type of meter, and consists of a horizontal wheel carrying a series of cups that rotate on a vertical axis. Half of the cups behave convex and half as concave to the current. There is a tail-vane and a counter-weight at bottom, to balance the meter and to keep it steady, as shown in Fig. 7.70.

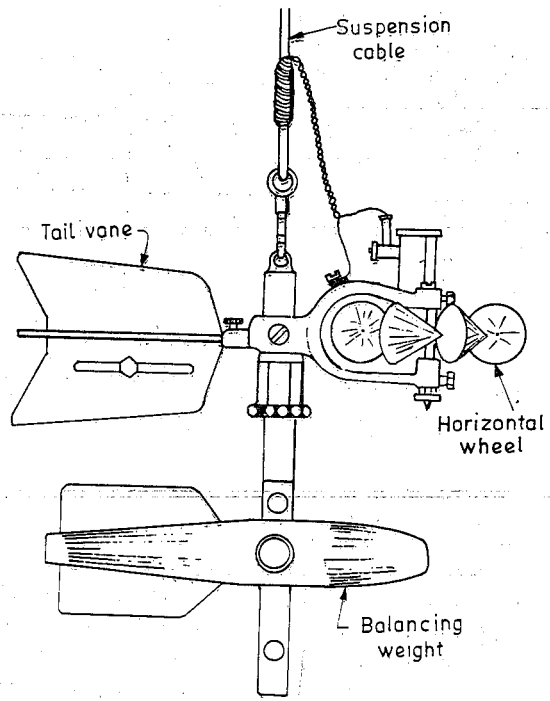


Fig. 7.70. Price's current meter.

When the current-meter is suspended in water, the velocity of flow causes the wheel to rotate. In this type of a meter, the difference in pressure produces the rotation; that is why, it is, called a *differential meter.* In another type of a meter, called a *direct meter,* the current acts directly and uniformly on all vanes, causing rotation.

The current meter is fitted with a device, so as to record the number of revolutions of the horizontal wheel due to the velocity of flow. In old models, telephone circuit operated by a battery, carries the signal to a telephone receiver at the ear of the observer, who can count the number of signals received. But in modern instruments, electro-magnetic counters, using digital or analog displays, are provided, eliminating the headache of counting. Knowing the number of revolutions per second made by the wheel, when suspended in water from a cable, etc., the velocity of flow can be easily calculated by using the **rating formula** of the given current meter. The **rating formula** is generally of the form:

$$V = (a + b\,N) \qquad \text{...(7.71)}$$

where V is the velocity of flow in m/sec.

N = No. of revolutions made by the wheel per second,

a and b are constants given by the manufacturer, or generally determined by experiments.

Since the velocity may be different for different compartments, the meter is lowered from a boat stationed in the centre of each compartment separately and individually. The wheel of the meter should face the current, and is generally kept at a depth of $0.6\,y$ from the free water surface, where y is the depth of water at that section.

The velocity measured by keeping the meter at a depth of $0.6\,y$ gives the mean velocity in vertical. Sometimes, the velocities at depths of $0.2\,y$ and $0.8\,y$ are determined, and their average value is taken to be the mean velocity.

The biggest advantage of the differential meters is that they have vertical shafts and, therefore, operate on point bearings with minimum amount of friction. These bearings must be protected from damage, and therefore, they are dismantled and carefully dried and oiled using a high grade watch oil. The process is repeated after every measurement, and also at the time of keeping the instrument in store.

For narrower streams, the meter can be suspended by a cable, stretched across the river, and the mid point of each compartment can be marked by a tag on this rope. But for wider streams, where it is difficult to stretch a wire rope across the river, two theodolites are installed, one on each bank, and are used for locating the centre of each compartment. The man in the boat will align his boat, till he finds himself in the centre of the compartment. The man will then measure the depth of water and will lower the current meter upto the required depth in order to measure the velocity.

The boat can be conveniently used for small velocities of the order of 1 m/sec, but for higher velocities and rough waters, it becomes difficult to keep the boat steady enough to permit measurements. In such a case, the boat is generally anchored.

The meter should then be suspended from a boom, projecting a few metres from the boat, so that the measured velocities are not affected by the presence of the boat.

Examples 7.18 and 7.19, solved below, explain the use of current meter measurements for computing streamflow.

Example 7.18. *Following velocities were recorded in a stream with a current-meter.*

Depth above bed (m)	*0*	*1*	*2*	*3*	*4*
Velocity m/sec	*0*	*0.5*	*0.7*	*0.8*	*0.8*

Find the discharge per unit width of stream near the point of measurement. Depth of flow at the point was 5 m.

(A.M.I.E., 1976)

Solution. $y = 5$ m

$\therefore$ $0.2\,y = 1$ m and $0.8\,y = 4$ m

Velocity measured by the meter when immersed at a depth of 0.2 y (*i.e.* 1 m from water surface or 4 m above bed) = 0.8 m/sec. Similarly, the velocity measured by the meter when immersed to a depth of 0.8y (*i.e.* 4 m from water surface or 1 m above bed) = 0.5 m/sec.

$$\text{Av. mean velocity} = \frac{0.8 + 0.5}{2} = 0.65 \text{ m/sec.}$$

The discharge per unit width of stream

$$= a \cdot V = (5 \times 1)\, 0.65 = 3.25 \text{ cumecs/m.} \quad \textbf{Ans.}$$

Example 7.19. *Compute the streamflow for the measurement data given below :*

Distance (m)	*0*	*0.6*	*1.2*	*1.8*	*2.4*	*3.0*	*3.6*	*4.2*	*4.8*	*5.4*	*6.0*	*6.6*
Depth (m)	*0*	*0.3*	*1.29*	*2.16*	*2.55*	*2.22*	*1.68*	*1.41*	*1.05*	*0.63*	*0.42*	*0*
Velocity (m/sec) at 0.2 d	*0*	*0.42*	*0.57*	*0.78*	*0.87*	*0.81*	*0.75*	*0.69*	*0.63*	*0.54*	*0.45*	*0*
Velocity (m/sec) at 0.8 d	*0*	*0.21*	*0.36*	*0.54*	*0.60*	*0.30*	*0.51*	*0.45*	*0.39*	*0.33*	*0.30*	*0*

(U.P.S.C. Engg. Services, 1967)

Solution. While analysing **standard current meter observations,** the river X-section is divided into segments, as shown in Fig. 7.71 (*a*). Each end segment consists of the triangular area, shown dotted, and the rest of the segments are rectangular, shown hatched.

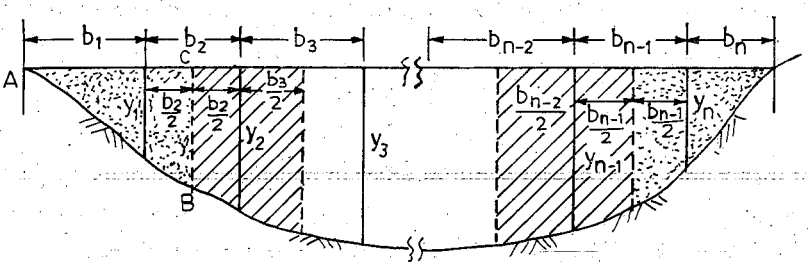


Fig. 7.71 (*a*) River section divided into n segments.

The X-sectional area of each intermediate rectangular segment can be easily determined by multiplying average widths, such as $\frac{b_2 + b_3}{2}$, $\frac{b_3 + b_4}{2}$ ..... $\frac{b_{n-2} + b_{n-1}}{2}$, with the

corresponding depths $y_2, y_3, \ldots\ldots y_{n-1}$. The area of each end segment can be calculated by mid point rule of a triangle, as :

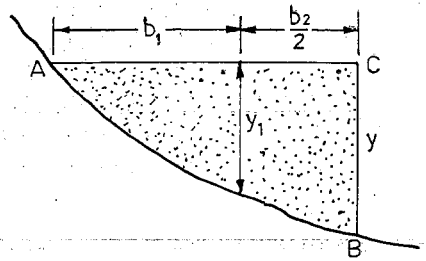


Fig. 7.71 (*b*) Larger view of left end segment.

$$\frac{b_1}{y_1} = \frac{b_1 + \frac{b_2}{2}}{y} \quad \text{(Refer Fig. 7.71}b\text{)}$$

$$\therefore \quad y = \frac{\left(b_1 + \frac{b_2}{2}\right)}{b_1} \cdot y_1$$

$$\therefore \text{ Area } ABC = \frac{1}{2}\left(b_1 + \frac{b_2}{2}\right) \cdot y$$

$$= \frac{1}{2}\left(b_1 + \frac{b_2}{2}\right)\left(\frac{b_1 + \frac{b_2}{2}}{b_1}\right) y_1 = \frac{\left(b_1 + \frac{b_2}{2}\right)^2 y_1}{2b_1}$$

In other words, the effective width for left end segment, considered as a rectangle of depth y_1, will be

$$\frac{\left(b_1 + \frac{b_2}{2}\right)^2}{2b_1} \qquad \ldots(7.72)$$

Similarly, the effective width for the right end segment will be equal to

$$\frac{\left(b_n + \frac{b_{n-1}}{2}\right)^2}{2b_n} \qquad \ldots[7.72\ (a)]$$

Using the above theory, in the given numerical example, we have the effective width of the first segment

$$= \frac{\left(b_1 + \frac{b_2}{2}\right)^2}{2b_1} = \frac{\left(0.6 + \frac{0.6}{2}\right)^2}{2 \times 0.6}$$

$$= 0.675 \text{ m}$$

Since $b_1 = b_2 = b_3 = \ldots. b_9 = b_{10} = 0.6$ m ($n = 10$), we have

width of the other end segment (9th) = width of the first end segment = 0.675 m

Av. width of each intermediate segment (2nd to 8th) = 0.6 m

The calculations for computing segmental discharges, through all the 9 segments, can now be carried out in a tabular form, as shown in Table 7.27.

Table 7.27

S. No.	Distance from one edge (b) m	Average effective width m	Depth(d) m	Velocity in m/sec			Segmental discharge ΔQ $(3)\times(4)\times(7)$ m^3/sec
				at 0.2d	at 0.8d	mean vel. $\frac{(5)+(6)}{2}$	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	0	0	0	0	0	0	0
1	0.6	0.675	0.3	0.42	0.21	0.315	0.064
2	1.2	0.6	1.29	0.57	0.36	0.465	0.360
3	1.8	0.6	2.16	0.78	0.54	0.66	0.855
4	2.4	0.6	2.55	0.87	0.60	0.735	1.125
5	3.0	0.6	2.22	0.81	0.30	0.555	0.739
6	3.6	0.6	1.68	0.75	0.51	0.645	0.650
7	4.2	0.6	1.41	0.69	0.45	0.57	0.482
8	4.8	0.6	1.05	0.63	0.39	0.51	0.321
9	5.4	0.6	0.63	0.54	0.33	0.435	0.164
10	6.0	0.675	0.42	0.45	0.30	0.375	0.106
11	6.6	0	0	0	0	0	0
							$\Sigma = 4.867$ m^3/sec

The sum total of all the segmental discharges, computed in column (8) of Table 7.27, works out to **4.867 cumecs,** as the total discharge of the stream. **Ans.**

7.35.5.1. Gauge discharge curves. The discharge of a stream is generally calculated, as explained above, at different water levels in the stream. These water levels, called **gauge heights,** and the corresponding discharges are, thus, recorded, and plotted to obtain a curve, known as **discharge curve** or **station rating curve,** as shown in Fig. 7.72. When once, such a curve has been drawn, it is very easy to find out the discharge

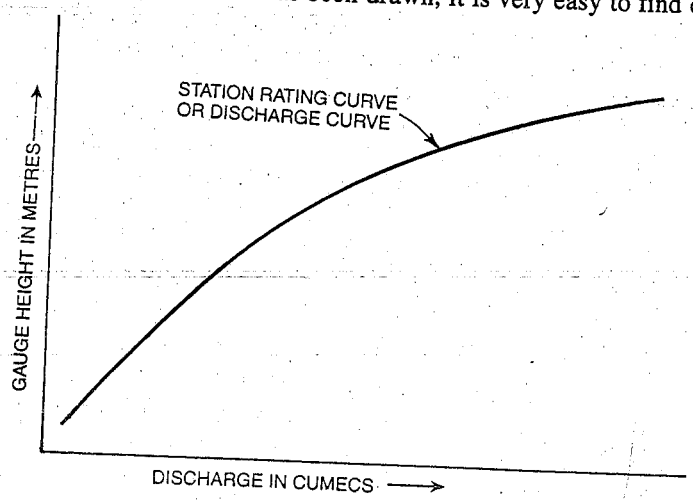


Fig. 7.72. A typical Gauge-discharge curve.

in the stream, simply by reading the river gauge, and reading out the corresponding discharge from the standard gauge discharge curve.

Such a curve gives us a relationship between the stage of the river at a given time *(i.e.* gauge ht.) and the corresponding discharge, and is hence known as **stage-discharge curve.** The relation expressed by this curve is known as stage discharge relationship at the given *velocity-area station.*

A velocity area station essentially possesses : *(i)* a *control, (ii) a gauge;* and *(iii) a metering section.*

The **control** or **control section** is that particular cross-section of the river which helps us in determining the relationship between the stage of the river at any time and the discharge in the river at that time. A control may be a natural or an artificial one. It may be a permanent control or a shifting control.

Due to the presence of a control across a river, the water on the upstream side gets affected. The distance upto which the flow gets affected due to the presence of the control may be called as the *length of influence of the control.* Natural undulations in the river bed may affect the water surface, and hence may act as controls. Depending upon the stage (low or high) of the river, a particular undulation may behave as a control, while the same undulation may not behave as a control for some other stage of the river. In that stage, some other undulation may start behaving as a control. Hence, such an undulation which does not behave as a control for almost all practically involved river stages, is called a **shifting control,** while if an undulation produces almost the same effect on the water surface for almost all the river stages, then the undulation may be termed as a **permanent control.** The cost of obtaining the discharge records mainly depends upon the permanency of the control. The location of the control must, therefore, first be decided with due care and considerable thought, so as to obtain as permanent a control section as possible. Broadly speaking, the control section must be in a straight portion of the river, which is relatively less erodible.

In fact, it is not possible to obtain an absolutely permanent control. The continuous flow of water and continuous temperature changes cause even the hardest rocks to erode and disintegrate slowly. However, a control made up of hard resistant rock, may behave as a permanent control over long periods.

After location of the control section has been determined, the gauge should be installed. The gauge is an instrument that is installed upstream of the control section but within its influence. The gauge helps in determining the fluctuations in the river stage with respect to time. The gauge readings (measured from the bottom as depth of flow or in GTS levels as elevation) represent the stage of the river at different times. These readings are sometimes required to be corrected before plotting them against discharges. A gauge may be of a recording type or may be of a non recording type. Recording gauges are automatic recording gauges and have the advantage over non recording gauges in the sense that they do not involve manual mistakes, because a recording gauge has not to be read by an observer.

While fixing a gauge, it should be kept in mind that its support is rigid and immovable, so that the elevation of its datum is not likely to change w.r.t. time. The elevatior of its zero should be determined w.r.t. at least two permanent bench marks. After a gauge has been installed, its datum should be kept unchanged. The zero of the gauge must be checked annually, and the gauge must be reset as soon as a change in its datum is detected. Unless this precaution is taken, long periods of records are likely to be doubtful, and it is not possible to correct them afterwards.

7.35.5.1.1. Effect of changing stage on the stage discharge curve. The discharge measured for a particular given stage is different for a changing stage than for a constant stage. The stage of a river may be constant, may be rising, or may be falling. *During the rising stage of the river, the measured discharge is more than that for a constant stage. Similarly, during a falling stage of the river, the measured discharge is less than that for a constant stage.*

While plotting a stage-discharge curve, the values of discharges (to be plotted against the values of stages) must be taken for a constant stage. But this is not the practical case, and the values of discharges are generally measured either during a rising stage or during a falling stage. Hence, to obtain a true discharge curve, the measured discharges must be corrected for the changing stage.

During a rising stage, the measured discharge is more than the true discharge for a given gauge height. In order to make this observed point fall on the true discharge curve, either the discharge should be reduced or the gauge height should be increased. Hence a –ve correction should be applied to the discharge value, or a +ve correction to the gauge reading for a rising river-stage. The case is reversed for a falling river stage. The true discharge curve and the observed discharge curves are shown in Fig. 7.73.

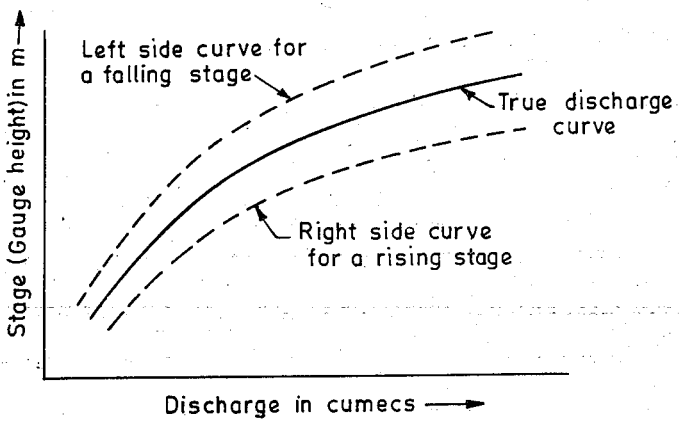


Fig. 7.73. True discharge curve *Vs* Observed discharge curve.

The results of discharge measurements for a *rising* stage fall to the *right* side, and that for a falling stage to the left side of the true discharge curve, as shown in Fig. 7.73.

The amount of deviation from the true discharge curve does not depend upon the total rise or fall during a particular set of measurements, but depends only on the rate at which this change occurs. Thus, the deviation of the observed values from the true discharge values depends upon the rate of change of stage, and, therefore, while checking or drawing a discharge curve, the rate of change of stage must be computed. Time should, therefore, be recorded at the beginning and at the end of the gauging, in order to compute the rate of change of stage.

A graph can be plotted between the rate of change of stage (X-axis) and the percentage correction to be applied to the observed discharge values, in order to obtain true discharges (Y-axis). This relationship can be plotted separately for rising stage as well as for the falling stage. Generally, the relation found during a rising stage is different from that obtained during a falling stage, and usually, the relation is found to be a straight line or a very flat curve, as shown in Fig. 7.74.

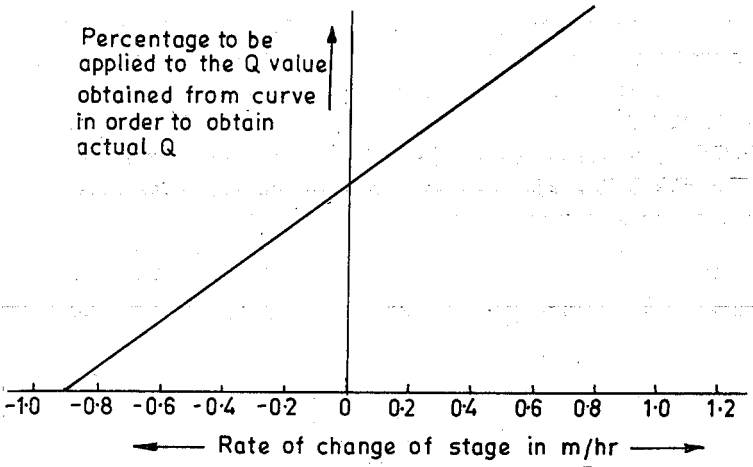


Fig. 7.74

Such curves are plotted carefully and kept for future use. The rate of rise or fall during periods of sudden change is always known. The percentage correction to be applied can be read from the graph and these corrections (+ ve for a falling stage and –ve for a rising stage) can be applied to the observed discharge values.

7.35.5.1.2. Effect of shifting control on stage-discharge curve. We have already stated earlier that the permanency of the control must be fully ensured while fixing a velocity area station. But a control does not always behave as a permanent control, but changes its position many times. Due to this shift or a change in the control, the observed discharge measurements are not the true values, and will not fall on a true discharge curve.

For a fixed control with a changing stage, the observed points should fall on the *right* side of the true discharge curve for a *rising* stage, and the points should fall on the left side of the true discharge curve for a falling stage.

But if on plotting the observed discharge values, it is found that the points neither fall on the true discharge curve nor fall in accordance with the above provisions for changing stage; then we must at once feel that either some observation mistake has taken place or the control has shifted away.

In other words, if for a rising stage, the observed points fall to the left side of the true discharge curve ; or if for a falling stage, the observed points fall to the right side of the true discharge curve; or for a constant stage, the observed points do not fall on the true discharge curve; then we must at once become alert and should think of the possibility of a change in the control.

If a control is found to be changing, the next step is to find out the speed with which the control is changing. The rate at which the control is changing is the only important factor, which decides the method to be adopted for correcting the measurements.

If the control is changing very slowly or only at the time of floods, then the best method to deal with the problem is to draw a new discharge curve (or a rating curve) as soon as sufficient change in the control is detected. This new curve is then applied for a period till sufficient deviation due to the change in control is again detected. On an average, a control changes sufficiently once a year. *In other words, the rating curve has to be changed, almost once a year.*

But on many other rivers, the control changes quite rapidly and constantly. In such cases, **Stout's method** is generally used to correct the observed discharge measurements. In this method, daily discharge records are obtained and corrected, as explained by the following example:

Example 7.20. *The gauge heights and a few discharge measurements of a stream are given in the table below. Plot the stage-discharge curve and tabulate the corrected gauge heights and the corresponding discharges.*

Date	*Gauge height in m*	*Discharge in cumecs*
(1)	*(2)*	*(3)*
July, 1971		
1	*2.32*	*268*
2	*2.31*	
3	*2.28*	
4	*2.25*	
5	*2.17*	
6	*2.07*	
7	*1.92*	
8	*1.83*	*168*
9	*1.62*	
10	*1.41*	
11	*1.23*	*98*
12	*1.09*	
13	*1.00*	
14	*1.06*	
15	*1.16*	
16	*1.30*	*121*
17	*1.52*	
18	*1.46*	
19	*1.36*	
20	*1.21*	
21	*1.09*	*80*
22	*0.97*	
23	*0.90*	
24	*0.80*	
25	*0.75*	
26	*0.70*	
27	*0.66*	
28	*0.69*	*47*
29	*0.71*	
30	*0.80*	
31	*0.40*	*24*

(U.P.S.C., Engg. Services, 1972)

Solution. Fig. 7.75 shows the discharge curve, as obtained from the given measurements. The exact manner in which this curve is drawn is not of much importance. The

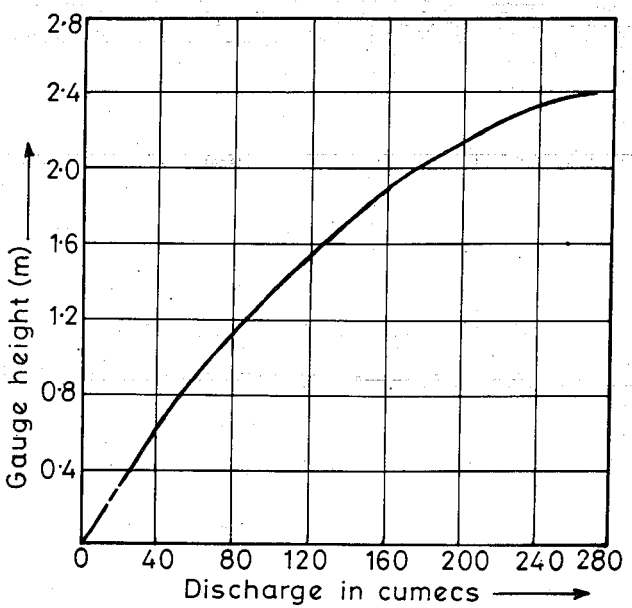


Fig. 7.75. Gauge discharge curve or Rating curve.

corrected gauge heights and, therefore, the corrections corresponding to these discharges are found and written in brackets in column (4) and column (5) of table 7.28, in such a way, that the discharge values as obtained from this curve are the same as the measured discharges. These values are plotted in Fig. 7.76 and a curve is drawn through these points, so as to obtain what is known as gauge-height correction curve.

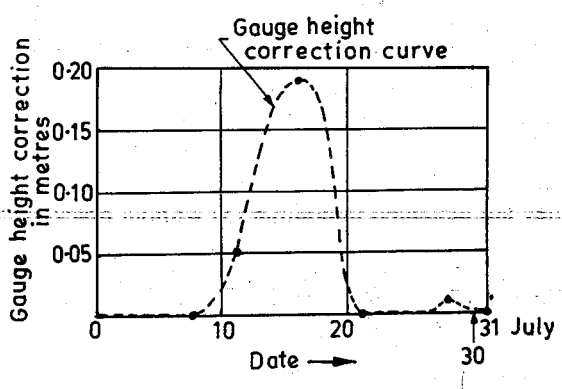


Fig. 7.76. Gauge-height correction curve.

Table 7.28

Date	*Gauge ht. in metres (given)*	*Given discharge in cumecs*	*Corrected gauge hts. in metres for the pts. to be on the drawn curve*	*Gauge ht. corrections from G.H.C. curve of Fig. 7.76*	*Corrected gauge hts. for all the days*	*Discharge in cumecs*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1971 July						
1	2.32	268	**(2.32)**	**(0.00)**	2.32	268
2	2.31			0.00	2.31	260
3	2.28			0.00	2.28	244
4	2.25			0.00	2.25	236
5	2.17			0.00	2.17	216
6	2.07			0.00	2.07	200
7	1.92			0.00	1.92	176
8	1.83	163	**(1.83)**	**(0.00)**	1.83	163
9	1.62			+0.02	1.64	137
10	1.41			+0.035	1.445	116
11	1.23	98	**(1.28)**	**(+0.05)**	1.28	98
12	1.09			+0.08	1.17	87
13	1.00			+0.09	1.09	80
14	1.06			+0.12	1.18	88
15	1.16			+0.15	1.31	101
16	1.30	121	**(1.49)**	**(+0.19)**	1.49	121
17	1.52			+0.18	1.80	158
18	1.46			+0.16	1.62	135
19	1.36			+0.09	1.45	116
20	1.21			+0.02	1.23	23
21	1.09	80	**(1.09)**	**(+0.00)**	1.09	80
22	0.97			0.00	0.97	70
23	0.90			0.00	0.90	64
24	0.80			0.00	0.80	55
25	0.75			+0.005	0.755	51
26	0.70			+0.005	0.705	48
27	0.66			+0.01	0.67	44
28	0.69	47	**(0.70)**	**(+0.01)**	0.70	47
29	0.71			+0.01	0.72	49
30	0.70			+0.005	0.705	48
31	0.40	24	**(0.40)**	**(0.00)**	0.40	24

Gauge height corrections are read from the gauge height correction curve for the intervening days. They are tabulated in column (5) without brackets. These corrections with their appropriate signs (+ ve or –ve) are applied to the recorded gauge heights. The corrected gauge heights for the entire month are tabulated in column (6). Corresponding to these corrected heights, the discharge values are read from the rating curve drawn previously in Fig. 7.75. These discharge values are tabulated in column (7) of Table 7.28. The table is otherwise self explanatory.

The above example explains the **Stout's method** for obtaining the daily discharges. The gauge height correction curve was found to have only + ve values for the whole month in the above particular example, but it may not be so in some other case. In that

case, a proper curve having + ve as well as –ve values, plotted on both sides of X-axis, should be drawn.

Discharge observations are taken for a particular month and corrected as above. The discharge observations of subsequent months at some suitable intervals are taken and corrected, using the same discharge curve as long as the observed discharges continue to fall on both sides of the curve. However, if it is felt that some other curve can fit the points better, then another curve may be drawn, as explained earlier.

Extending the stage discharge curve. With an observed set of gauge discharge measurements, it is generally not possible to draw a complete curve. For very low discharges as well as for very high discharges, the curve has to be extended back and far, respectively.

Three methods, which are generally adopted to extend the discharge curves, are explained below:

(1) **Logarithmic method of extending discharge curve.** If the river cross-section at the site of the gauge is a uniform section (or even approximates a uniform section) to which can be fitted either a segment of a circle or of a parabola or a rectangular or a trapezoidal section, then this method can be easily adopted. The stage-discharge curve for such a stream can then be expressed by an equation of the form:

$$Q = K(h - h_0)^m \qquad ...(7.73)$$

where Q = discharge in cumecs

h = gauge height in metres.

h_0 = gauge ht. in metres corresponding to zero discharge

K and m are constants.

Taking log on both sides, we get

$$\log Q = \log K + m \log (h - h_0).$$

or
$$\log Q = m \log (h - h_0) + \log K \qquad ...(7.74)$$

This is the equation of a st. line having a slope as m and $\log K$ as the intercept on Y-axis.

Generally, the gauge height corresponding to zero discharge is not known, and therefore, it is not easy to extend the curve by using this equation. Hence, a graph is plotted between $\log Q$ and $\log (h - h_0)$ for different assumed values of h_0 other than zero, till a st. line is obtained for a certain value of h_0. After knowing h_0, and by drawing this st. line, the equation of the discharge curve can be established, and hence the curve can be extended easily.

Another simpler method for evaluating h_0 is given by Prof. Running and is explained below:

Three values of discharge (Q_1, Q_2, Q_3) are selected from the known portion of the curve. One of these values should be near the middle of the curve, and the other value should be near the upper end of the curve. Then the third intermediate value should be so chosen that all the three values happen to be in geometrical progression, *i.e.*

$$Q_2 = \sqrt{Q_1 Q_3} \qquad ...(7.75)$$

For example, if two values of discharge are chosen to be 20 and 320, then an intermediate value equal to 80 can be chosen. All the three values of 20, 80, 320 are in

G.P. *i.e.* a, ar, ar^2. Let these three values be represented by points a, b and c respectively. Vertical lines are drawn through a and b, and horizontal lines are drawn through b and c, so as to intersect the verticals at d and e respectively, as shown in Fig. 7.77.

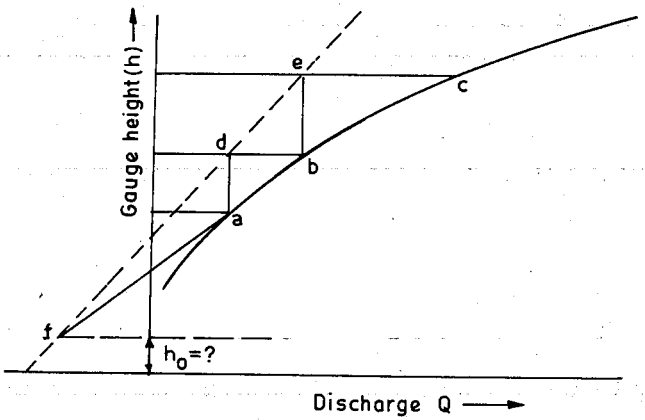


Fig. 7.77

A st. line ed is drawn through e and d which intersects the line ab (join of a and b) at point f. This pt. represents the elevation of the gauge ht. at zero discharge. Hence the elevation of this pt. gives the value of h_0. This method is based upon the assumption that the lower portion of the discharge curve containing these three pts. (*i.e.* a, b and c) all lie on a parabola.

If h_1, h_2 and h_3 represent the gauge heights corresponding to points a, b and c (*i.e.* discharges Q_1, Q_2 and Q_3), then mathematically, h_0 can be given by

$$h_0 = \frac{h_1 h_3 - h_2^2}{h_1 + h_3 - 2h_2} \qquad \text{...(7.76)}$$

By using the above equation, h_0 can be very easily determined.

Also from equation (7.73), we can write

$$\frac{Q_1}{Q_2} = \left(\frac{h_1 - h_0}{h_2 - h_0}\right)^m \qquad \text{...[7.73 (}a\text{)]}$$

Using the known values of Q_1, Q_2, h_1, h_2 and h_0, the value of m can be easily determined, and then from equation (7.73), the value of constant K can be found, thus determining the equation of the rating curve.

(2) **A $\sqrt{y}$ method for extending the discharge curve.** This method was devised by Steven, and hence known after him as Steven's method. The method is applicable for wider and shallower streams. For such streams, the wetted perimeter P (*i.e.* $P = b + 2y$) is approximately equal to b ($\because y << b$). Therefore, hydraulic mean radius $R\left(i.e.\ R = \frac{A}{P}\right)$ is equal to $\frac{A}{b} = y$.

$\therefore$ $R = y$ for such wider and shallower streams.

Now $Q = C \,.\, A \,.\, \sqrt{R \,.\, S}$ by Chezy's formula.

or $Q = C \,.\, A \,.\, \sqrt{R}\,\sqrt{S}$

or $Q = C \cdot A \cdot \sqrt{y}\sqrt{S}$ for shallow and wider streams

or $Q \propto A\sqrt{y}$ if $C\sqrt{S}$ is constant

or $Q = k \cdot A \cdot \sqrt{y}$. ...(7.77)

But A and y are both dependent upon gauge height (h) and are also subject to determination by field survey ; and hence a graph (curve 1) can be plotted easily for values of $A \cdot \sqrt{y}$ for different values of gauge height (h), as shown in Fig. 7.78. Then a graph (curve 2) is plotted between observed Q and values of $A\sqrt{y}$ ($A\sqrt{y}$ values are either given *or* determined from the first curve for the observed values of h).

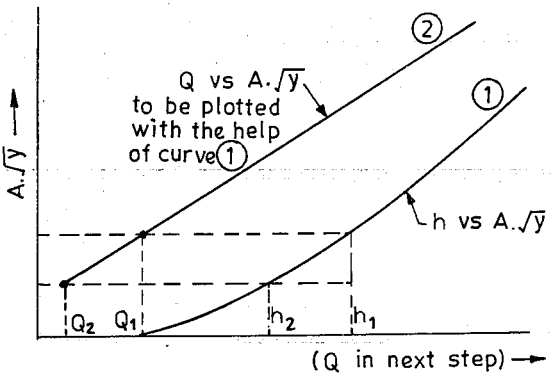


Fig. 7.78

The graph between Q and $A\sqrt{y}$ is found to follow a st. line, if $C\sqrt{S}$ remains constant. This st. line can be extended to evaluate discharge for any higher value of $A\sqrt{y}$ corresponding to any higher value of h (*i.e.* h value being beyond the given rating curve). For this higher value of h, for which Q is reqd., the value of $A\sqrt{y}$ can either be determined by field survey, or if no such data is available, the h vs $A\sqrt{y}$ curve may be extended to approximate the value of $A\sqrt{y}$ for that higher value of h.

Example 7.21. *Data for the stage-discharge relation for a gauge station on a stream are given below:*

Stage (h) *m*	*Area (A)* m^2	*Av. depth (y)* *m*	*Discharge (Q) cumecs*
0.52	*24.4*	*0.46*	*28.9*
1.06	*111.5*	*0.64*	*138.8*
1.30	*166.3*	*0.98*	*218.0*
1.71	*221.1*	*1.40*	*303.0*
2.04	*304.7*	*1.58*	*427.6*
2.38	*367.9*	*1.74*	*538.1*
2.81	*464.5*	*1.86*	*708.0*
4.42	*761.8*	*2.74*	?

Based on Chezy's formula and using average depth (y) *instead of R, discharge* $Q = k \cdot A \cdot A\sqrt{y}$, *plot* Q *vs.* $A\sqrt{y}$, *and extend the line and obtain* Q *at the stage of 4.42 m.*

Solution. First of all, the values of $A \cdot \sqrt{y}$ are determined for different values of Q from the known values of A and y. They are tabulated in table 7.29.

Table 7.29

h	A	y	Q	$\sqrt{y}$	$A\sqrt{y}$
0.52	24.4	0.46	28.9	0.68	16.55
1.06	111.5	0.64	138.8	0.80	89.20
1.30	166.3	0.98	218.0	0.99	164.63
1.71	221.1	1.40	303.0	1.18	261.61
2.04	304.7	1.58	427.6	1.26	383.00
2.38	367.9	1.74	538.1	1.32	485.29
2.81	464.5	1.86	708.0	1.36	633.50
4.42	761.8	2.74	?	1.66	1261.00

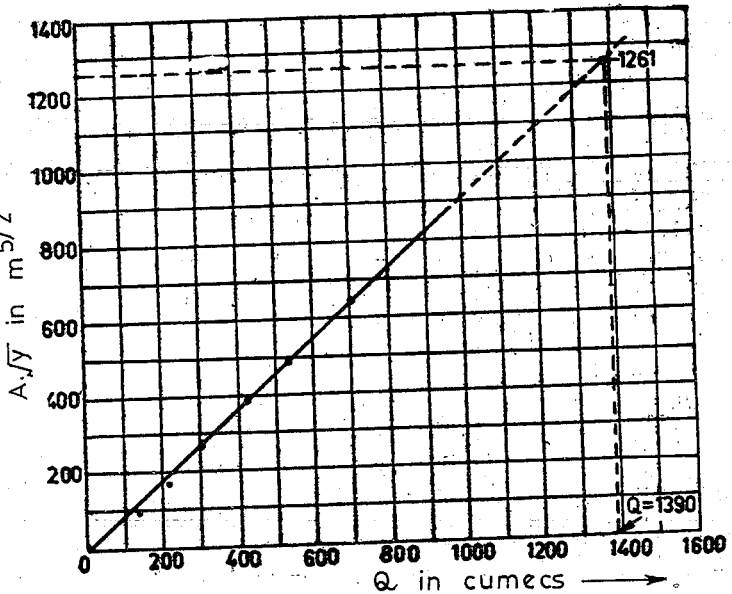


Fig. 7.79

A graph is now plotted between Q (on x-axis) and $A\sqrt{y}$ (on y-axis), as shown in Fig. 7.79. The st. line drawn is extended, so as to determine Q for the value of $A\sqrt{y}$ equal to 1261; and reqd. Q works out to **1390 cumecs. Ans.**

(3) **General method for extending the discharge curve.** This method is applicable to all types of river sections, but it is necessary to know the stream cross-section at the

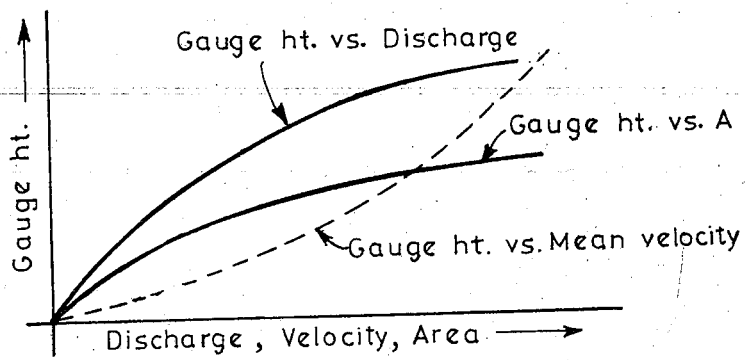


Fig. 7.80

gauge site. The cross-section areas for different gauge heights can then be worked out by a planimeter, and a curve is plotted between gauge and area (gauge *vs.* area curve) on the gauge-discharge curve itself. From these two curves, the velocity curve can also be drawn $\left(\text{mean velocity} = \frac{\text{discharge}}{\text{area}}\right)$ on the same drawing. The mean velocity curve is generally found to be concave upward, and the area curve is generally found to be convex upward, as shown in Fig. 7.80.

After having determined the mean velocity curve, the problem left is only to extend the same. We know that velocity is given by Manning's formula, as:

$$V = \frac{1}{n} \cdot R^{2/3} S^{1/2} \qquad ...(7.78)$$

But $\frac{1}{n} \cdot S^{1/2}$ is almost constant for higher river stages after all the rapids have become drowned out; and we may, therefore, write

$$V \propto R^{2/3}$$

or
$$V = KR^{2/3} \qquad ...(7.79)$$

where K is a constant and R is the hydraulic mean depth

With the cross-section and area known for all stages, R can be worked out for all stages. By taking various values of K from the known portion of the velocity curve, and the corresponding values of R, the values of K can be calculated for various available gauge heights *(i.e.,* in the range in stage for which mean velocity curve is known). These values of K are now plotted against gauge heights, so as to obtain a curve which should approach a vertical line as an asymptote in the higher stages. By extending this line, we should obtain values of K quite accurately, which when combined with their respective values of R and A, will give values of discharges that may be used for extending the discharge curve for all higher stages.

7.35.6. Computation of River Flow from Power Plant Records. Although, this is an indirect method for computing the discharge of a river, even then, this is the most accurate of all the methods, and may be adopted at places, where no direct method, such as *weir station, control station,* or *velocity area station,* is available.

Power plants are generally installed on the rivers, and their operation records help us in determining the mean daily discharge past those plants. The total flow passing through a power plant is the summation of flows through (*i*) turbines; (*ii*) over the spillways; (*iii*) through various sluices, gates, etc.; and (*iv*) through leakage.

The quantities of these partial flows can be computed and their summation will give us the total flow.

The discharge through turbines can be determined quite accurately these days. Their capacity for various gate openings are already calibrated; if not, they can be calculated by salt-solution method. After a turbine has once been rated for the various gate openings, its rating remains almost unaffected over years of service.

The discharge over spillway can be computed by using its appropriate formula. Similarly, the discharge through sluices and gates can be calculated by using their proper coefficients. They can be calibrated by current-meters, etc., if these coefficients are not available in the engineering literature.

The leakage through the dam may be computed by closing the turbines and other openings for short time during low-water, and then the total flow downstream from the plant can be measured with a current meter.

All these four quantities are added, so as to get the total daily discharge.

This method is quite useful and accurate, and especially during flood peaks, the peak discharge can be determined more accurately by this method than by either of the other methods.

7.35.7. Dilution Method of Measuring Discharge. This method of measuring discharge can be used for streams as well as for conduits.

The principle underlying this method is explained below:

When a solution of a **chemical** or a **tracer** is injected into a flow (pipe flow or stream flow), it will get completely and uniformly mixed with the flow, and that the diluted concentration downstream, will decrease due to increased discharge of the mix. Chemical concentrations are measured by *titration* or *colourimetric methods.* **Radioactive tracers,** when used, can be measured for radioactivity, by **Greiger counter.**

The streamflow (Q) can, thus, be computed by injecting a chemical flow (q) of concentration C_1 into the streamflow, and finally measuring the concentration C_2 of the mix at a certain downstream point. The equation, then, becomes

$$C_2 = \frac{C_1 q + 0 \times Q}{q + Q}$$

or

$$C_2 = \frac{C_1 q}{q + Q} \qquad ...(7.80)$$

When the added chemical flow q is quite small as compared to the streamflow Q, we can write $Q \approx Q + q$.

The eqn. (7.80), then becomes

$$C_2 = \frac{C_1 q}{Q}$$

or

$$Q = \frac{C_1}{C_2} \cdot q \qquad ...(7.81)$$

The above equation can, thus, directly compute the value of streamflow (Q), since C_1, C_2 and q are known.

This method is certainly of a high value in *hilly streams,* where measuring of gauges may not be feasible. This method is also quite useful in *waste water gaugings.*

The usual tracers, which are used in this method, are: **chemicals** like *Sodium chloride (common salt), Sodium dichromate, Manganese sulphate, Sodium nitrate, Lithium and Potasium salts;* **fluorescent dyes** like *Sodium fluoroscein, Rhodamine-WT, and Sulpho-Rhodamine B Extra;* and **radioactive isotopes** *like Bromine- 82, Sodium-24,* and *Iodine-132.*

The two **types of techniques** used in the above method are discussed below:

(1) **Constant Rate Injection Method.** In this technique, a concentrated solution of the tracer chemical is prepared at the gauging site in a tank, and is thoroughly mixed.

This chemical solution is now injected into the stream at a controlled constant rate (q), by using a device such as a *constant volume displacement pump* or a *constant head tank.* This rate (q) is measured by a flow meter in the line with a high degree of accuracy. A steady injection of chemical flow into the stream should be continued for a period equal to the time required in reaching steady mix conditions, plus the time of sampling at the downstream sampling point.

A highly turbulent and narrow reach is desirable. The reach should be long enough to ensure complete mixing of chemical flow and original river flow. This minimum required mixing length, can be roughly estimated by empirical equations, such as the one given by *Rimmar,* as :

$$L = \frac{0.13B^2\, C\,(0.7C + 2\sqrt{g})}{g \cdot y} \qquad ...(7.82)$$

where L = mixing length
B = average width of the stream
y = average depth of the stream
C = Chezy's coefficient of roughness, varying from 15 to 50 for smooth to rough bed conditions
g = acceleration due to gravity = 9.81 m/sec^2.

The value of L may vary from about 2 to 100 Km for a hilly stream, carrying discharge of about 1 cumec to 300 cumecs, respectively.

The concentration C_2 of the flow sample collected at the downstream point, will finally help in computing Q by using eqn. (7.81), as derived above.

(2) **Sudden Injection Method.** Sudden injection method, also called *Integration method,* is preferred for very large flow measurements in natural streams.

In this technique, a known volume V of the tracer chemical is introduced into the stream as quickly as possible. Sampling is carried out at a point sufficiently downstream, ensuring complete lateral mixing, for a period during which the tracer passes (which includes the complete injection cycle).

The stream discharge Q is then given by the equation

$$Q = \frac{V}{T} \cdot \frac{C_1}{C_2} \qquad ...(7.83)$$

where V = flow injected into the stream.
C_1 = Concentration of tracer into the injected flow.
T = total sampling time
C_2 = average concentration of tracer in water, sampled in a number of samples, during the entire sampling period (T).

The above equations will, no doubt, hold good only when the used tracer is not at all present in the natural streamflow. This technique, like the previous constant rate injection technique, does not require a constant rate injection pump, etc. and is a very simple method. The method does not require any calibration, and it is also not necessary

to measure the dimensions of the test section. This technique can also be used with radioactive tracers.

Example 7.22. *20 g/l solution of a chemical tracer was discharged into a stream @ 0.01 l/s. At sufficiently far downstream observation point, the chemical was found to reach an equilibrium concentration of 5 parts per billion. Estimate the stream discharge. The background concentration of the tracer chemical in stream water may be taken as Nil.*

Solution. Here $q = 0.01$ L/s

$$= \frac{0.01}{10^3} \text{ m}^3/\text{s} = 10^{-5} \text{ m}^3/\text{s}$$

$$Q = ?$$

$$C_1 = 20 \text{ g/L} = 20{,}000 \text{ mg/L} = 20{,}000 \text{ ppm}$$

$$= \frac{20{,}000}{10^6} = 2 \times 10^{-2}$$

$$C_2 = 5 \text{ parts per billion}$$

$$= \frac{5}{10^9} = 5 \times 10^{-9} \qquad (1 \text{ billion} = 10^9)$$

Now, using eq. (7.81), we have

$$Q = \frac{C_1}{C_2} \cdot q$$

$$= \frac{2 \times 10^{-2}}{5 \times 10^{-9}} \times 10^{-5} \text{ m}^3/\text{s}$$

$$= \mathbf{40 \text{ m}^3/\text{s}} \quad \textbf{Ans.}$$

7.35.8. Slope-Area Method for Computing Discharge of a River. It is a very versatile, though an indirect method, of stream flow estimation. It requires :

(*i*) selection of a stream reach in which cross-sectional properties including bed elevations are known at both ends;

(*ii*) the value of Manning's n; and

(*iii*) the water surface elevations at both ends of the stream reach.

The *Manning's formula* (*i.e.* resistance equation for uniform flow in an open channel) can be used to relate the depths at either ends of a river reach, to the discharge flowing in that reach, as follows:

Let the length of the given reach of a stream be L, and its bed slope be S_0. Let y_1 and y_2 be the water depths, and v_1 and v_2 be the velocities at the two ends, as shown in Fig. 7.81. Let A and B represent the water surface elevations at the two ends, which are known; and the discharge Q flowing through this stream reach is to be computed.

Applying Bernoulli's equation between the two sections (w.r.t. Fig. 7.81), we have

$$Z_1 + y_1 + \frac{v_1^2}{2g} = Z_2 + y_2 + \frac{v_2^2}{2g} + h_L \qquad ...(i)$$

where h_L = Head loss in the reach, which can be considered to be made up of two parts, *viz.*

(*i*) **frictional loss** h_f; and

(*ii*) **eddy loss** h_e

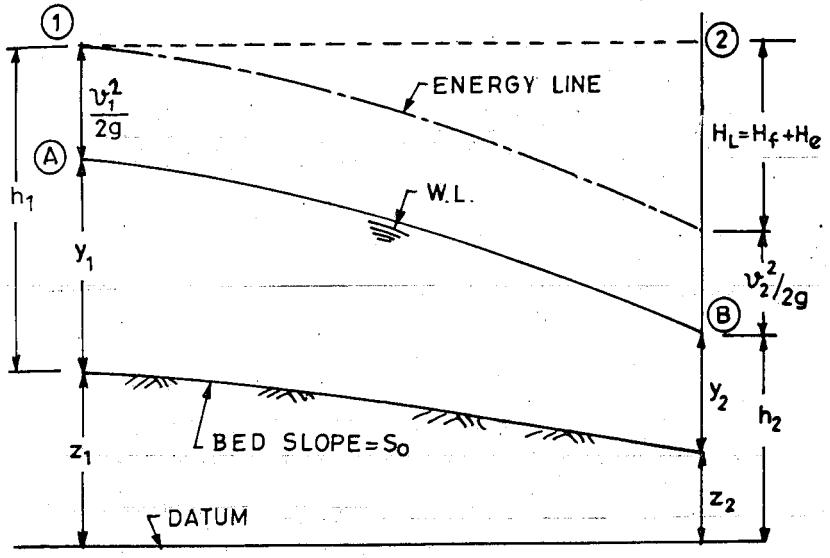


Fig. 7.81. Slope area method.

Denoting $z + y$ as h, being the water surface elevation above the datum, equation (*i*) can be written as :

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} + h_f + h_e \qquad ...(ii)$$

or

$$h_f = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - h_e \qquad ...(7.84)$$

From Manning's formula, we have

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S_f} \qquad ...(7.85)$$

where $\frac{1}{n} \cdot A \cdot R^{2/3} = K =$ **Conveyance factor** for the channel ...(7.86)

S_f is energy slope $= \frac{h_f}{L}$ ...(7.87)

$$\therefore \quad Q = K \cdot \sqrt{S_f} \qquad ...[7.85(a)]$$

For *non-uniform flow,* an *average conveyance factor* K_{av} is used to estimate the average energy slope, as :

$$\overline{S}_f = \frac{h_f}{L} = \frac{Q^2}{K_{av}^2} \qquad ...(7.88)$$

where $K_{av} = \sqrt{K_1 \cdot K_2}$ ...(7.89)

where $K_1 = \frac{1}{n_1} A_1 R_1^{2/3}$ ...(7.90)

and $K_2 = \frac{1}{n_2} A_2 R_2^{2/3}$ ...(7.91)

The **eddy loss** (h_e) is estimated as

$$h_e = K_e\left[\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right] \qquad ...(7.92)$$

where K_e = Eddy loss coefficient having values shown in table 7.30

Table 7.30. Eddy-loss Coefficient (K_e)

S. No.	*Cross-section characteristics of the channel reach*	*Value of K_e*	
		Expansion	*Contraction*
1.	Uniform	0	0
2.	Gradual transition	0.3	0.1
3.	Abrupt transition	0.8	0.6

Equations (7.84), (7.88), and (7.92) together with the continuity equation $Q = A_1v_1 = A_2v_2$ enable the discharge Q to be computed for known values of h, channel cross-sectional properties, and n.

The discharge (Q) infact, is computed by trial and error procedure, using the following steps :

(*i*) Assume $v_1 = v_2$. This leads to $\frac{v_1^2}{2g} = \frac{v_2^2}{2g}$ and by equation (7.84), $h_f = (h_1 - h_2) - h_e$. Ignore h_e, and calculate h_f from known water surface elevations at two points.

(*ii*) Use equation (7.85) and compute $Q = K \cdot \sqrt{\frac{h_f}{L}}$

(*iii*) Compute $v_1 = \frac{Q}{A_1}$, and $v_2 = \frac{Q}{A_2}$; $\frac{v_1^2}{2g}$ and $\frac{v_2^2}{2g}$, and h_e

(*iv*) Now, calculate the refined value of h_f by using the above values of $\frac{v_1^2}{2g}$, $\frac{v_2^2}{2g}$ and h_e in equation (7.84)

(*v*) Again calculate v_1, v_2, h_e, etc. and repeat the procedure till the two successive calculations give values of Q or h_f differing by a negligible margin.

7.35.8.1. Estimation of flood discharge by slope area method. This method can be very suitably used to estimate the peak discharge that has passed through an ungauged stream, from *high water marks* left by a flood event. The *high water marks* usually left by floods include; (*i*) *floating vegetation matter* such as grass, straw and seeds left stranded at high water levels when the flood subsides; (*ii*) *silt lines* left on river banks, including traces of erosion on the banks called *wash lines*; or (*iii*) silt or *stain lines* on buildings. Public enquiry from senior citizens of the area can also provide valuable information on high water marks. After cross-checking all such data, reliable data is retained about the peak river stages reached at different sections of the river. The slope-area method, described above, is then used to estimate the magnitude of the flood. The value of Manning's roughness coefficient is assumed based on the type and condition of the river. This method is expected to produce reliable approximate values of discharges.

The stream reach selected should, however, be straight and uniform, and the high water marks should be of good quality. If the stream is contracting or expanding, then a *contracting section should be preferred.* As an approximation, the reach length should be greater than 75 times the mean depth. The recorded fall in the water surface elevations based on high water marks should be greater than 0.15 m.

Example 7.23. *During a high flow, water surface elevations of a small stream were noted at two sections A and B, 10 km apart (A is upstream of B). These elevations and other salient hydraulic properties are given below:*

Section	*Water surface elevation (m)*	*Area of cross-section (m^2)*	*Hydraulic radius (m)*
A	104.771	73.293	2.733
B	104.500	93.375	3.089

The appropriate eddy loss coefficients are 0.3 for gradual expansion and 0.1 for gradual contraction. Estimate the discharge in the stream, assuming Manning's roughness coefficient, n, as 0.020.

[AMIE-1994 (W) New Scheme]

Solution. $L = 10\text{ km} = 10{,}000\text{ m}$

$h_1 = 104.771\text{ m}$ $\quad h_2 = 104.500\text{ m}$

$A_1 = 72.293\text{ m}^2$ $\quad R_1 = 2.733\text{ m}$

$A_2 = 93.375\text{ m}^2$ $\quad R_2 = 3.089\text{ m}$

$n = 0.020$

$K_e = 0.3$ (for expansion)

$= 0.1$ (for contraction)

$$K_1 = \frac{1}{n} \cdot A_1 R_1^{2/3}\text{; and } K_2 = \frac{1}{n} \cdot A_2 R_2^{2/3}$$

$$\therefore \quad K_1 = \frac{1}{0.020} \times 72.293 \times (2.733)^{2/3} = 7066\text{ m}^3/\text{s}$$

$$K_2 = \frac{1}{0.020} \times 93.375 \times (3.089)^{2/3} = 9903\text{ m}^3/\text{s}$$

Average K for the reach $= K_{av} = \sqrt{K_1 K_2} = \sqrt{7066 \times 9903} = \mathbf{8365}$

To start with, assume $v_2 = v_1$ and neglect h_e; then

$$h_f = (h_1 - h_2)$$

$$= 104.771 - 104.500 = 0.271\text{ m}$$

$$\bar{S}_f = \frac{h_f}{L} = \frac{0.271\text{ m}}{10{,}000\text{ m}}$$

$$\therefore \quad Q = K_{av} \cdot \sqrt{\bar{S}_f} = 8365 \sqrt{\frac{0.271}{10{,}000}} = \mathbf{43.55\ m^3/s}$$

Now, compute v_1, v_2, etc.

$$v_1 = \frac{Q}{A_1} = \frac{43.55}{73.293} = 0.594$$

$$\therefore \quad \frac{v_1^2}{2g} = 0.018$$

$$v_2 = \frac{Q}{A_2} = \frac{43.55}{93.375} = 0.466$$

$$\frac{v_2^2}{2g} = 0.011$$

$$h_e = K_e \cdot \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right)$$

$$= K_e\,(0.018 - 0.011) = 0.3^* \times 0.007 = 0.002$$

Using equation (7.84), we have

$$h_f = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - h_e$$

$$= 0.271 + 0.007 - 0.002 = 0.276$$

Repeating $\bar{S}_f = \frac{h_f}{L} = \frac{0.276}{10{,}000}$

$\therefore$ $$Q = 8365\sqrt{\frac{0.276}{10{,}000}} = 43.95\ \text{m}^3/\text{s}$$

$$v_1 = \frac{43.95}{73.293} = 0.600$$

$$\frac{v_1^2}{2g} = 0.018$$

$$v_2 = \frac{43.95}{93.375} = 0.471$$

$$\frac{v_2^2}{2g} = 0.011$$

Since there is no change in the newly computed *i.e.* refined values of $\frac{v_1^2}{2g}$ and $\frac{v_2^2}{2g}$, no further calculations are required, and hence the computed discharge of 43.95 m^3/s can be considered as the final figure. **Ans.**

RUNOFF AND FACTORS AFFECTING RUNOFF

Runoff is defined as the portion of the precipitation that makes its way towards rivers or oceans etc; as surface or subsurface flow. The discharge flowing in a river is the runoff from the basins drained by that river. Thus, the 'discharge', 'streamflow' and 'runoff' are generally used to mean one and the same thing.

7.36. Rainfall-Runoff Process Reviewed

Before we discuss the factors affecting runoff, and the computation techniques for runoff, let us, first of all, review the runoff process.

When rain falls, a part of it is intercepted by vegetation. Some of it is stored in depressions on the ground surface, and is known as **depression storage,** which later

*For expanding channel $K_e = 0.3$ and the present stream can be assumed to be expanding because A_2 as well as P_2 is more than A_1 and P_1.

infiltrates or evaporates. Some of the precipitation is absorbed by the soil, the amount of which depends upon the soil moisture conditions existing at the time of precipitation. If the rain continues further, the water starts infiltrating to the water-table, and if the rate of rainfall or the rate at which the water is reaching the ground (p) exceeds the infiltration rate (f), then this excess water starts collecting on the surface, as **surface detention**, and this water flows overland and joins the streams, rivers, lakes, oceans, etc. This flow is known as **surface runoff.** The water which percolates without joining the watertable, and then joins the stream as sub-surface flow, is known as *sub-surface storm flow,* and is considered as a part of surface runoff. On the other hand, the water that percolates to the ground watertable, and later after long periods, joins the river stream, is known as *ground water flow* or **base flow.**

The runoff, thus, actually consists of three portions:

(*i*) *surface runoff ;*

(*ii*) *base flow ; and*

(*iii*) *direct precipitation over the river stream.*

The second factor is important for the minimum flow of the river ; while the first factor is important for the maximum flow of the river ; and the third factor, being negligible, is generally ignored. For the peak flows, we are generally concerned with surface runoffs, and, therefore, many a times, the term runoff is exclusively used for surface runoff. The readers should not get confused, as the difference between them is quite clear.

7.37. Runoff Cycle

As defined earlier, the term *hydrologic cycle* is used to express the general circulation of water in its various forms from oceans to the atmosphere, to the ground from the atmosphere, and finally to oceans again. The *runoff cycle,* is a descriptive term applied to a part of this hydrologic cycle, *i.e.* the part between the precipitation from the atmosphere to the land areas, and its subsequent discharge through stream channels, or direct return to the atmosphere through evapo-transpiration.

The conditions existing at four different times, which are typical of the runoff cycle, are discussed below:

(1) **End of dry period and the beginning of a heavy isolated rainfall.** At this stage, all surface and channel storage, resulted from the previous rains gets depleted except for that in reservoirs, lakes and ponds. The only source of streamflow is the ground water flow, entering the river channel. The flow decreases with time, according to the storage depletion curve. Fig. 7.82 (*a*) shows the happenings occuring at this stage, which include evaporation (E), transpiration (T), and groundwater flow (G).

(2) **Shortly after beginning of rainfall and before interception and depression storage have been satisfied.** At this stage, a part of precipitation falls directly on the stream as channel precipitation (P), which becomes an immediate increment of streamflow. Part of the precipitation is intercepted (I) by vegetation and buildings, and so does not contribute to runoff. This stored water is eventually, returned to the atmosphere through evaporation.

Most of the water reaching the ground is either retained on the surface as depression storage (D) or passes through the soil as infiltration (F). Once it infiltrates below the soil, it starts replenishing the soil moisture deficiency, without adding to the ground water-

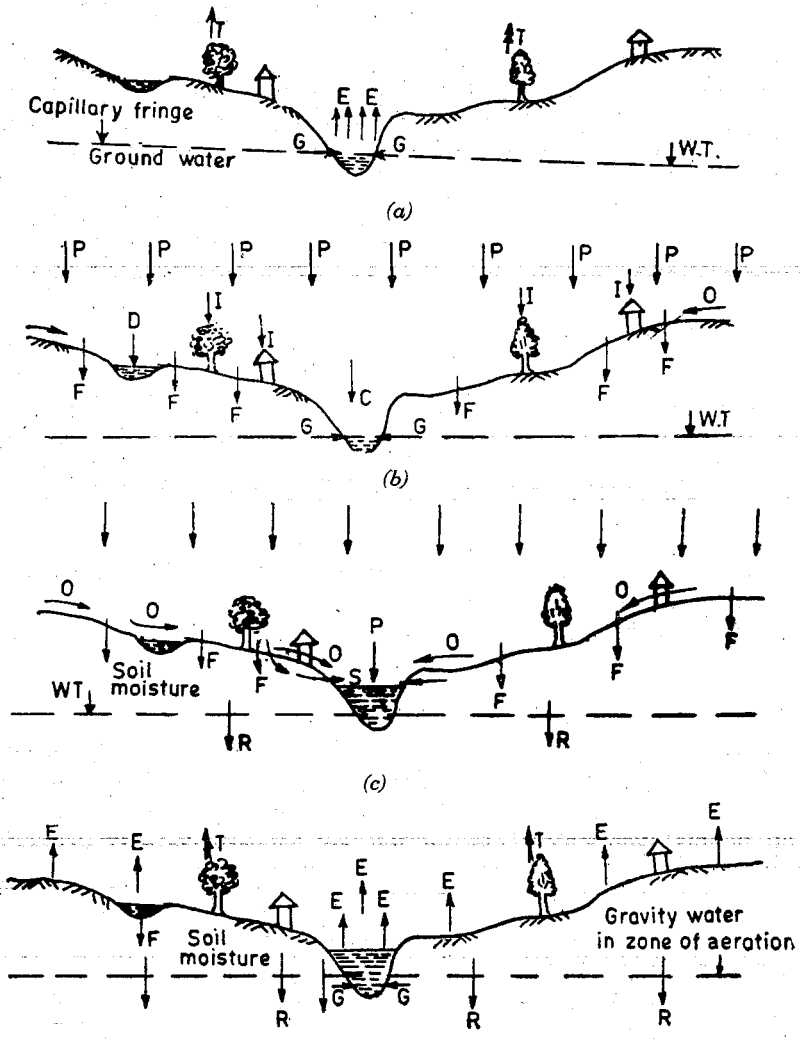


Fig. 7.82. Runoff cycle.

table. During this initial stage of rainfall, overland flow occurs only from small portions of the basin, such as impervious areas like roads, etc., and extremely steep slopes. From almost all other areas, the overland flow does not occur, as rain water gets consumed in losses, like interception, depression storage, and infiltration.

Rates of evaporation and transpiration, at this stage, are extremely low or negligible, as compared with those at fair weather conditions, because humidity is high, and because the evaporation capacity of the air tends to be satisfied by the falling rain rather than by soil moisture. The happenings taking place at this stage are illustrated in Fig. 7.82 (*b*).

(3) **Near the end of isolated heavy rainfall.** After many hours of heavy rainfall, virtually all depression storage and interception requirements get filled up, the soil

moisture deficiency is also satisfied to a considerable extent, and infiltration rate is near the minimum. The vegetation is saturated and rain falling on it is balanced by an equal amount falling from the vegetation to the ground, except for the extremely small quantity which returns to the atmosphere through evaporation. Similarly, the flow into the filled depressions is essentially balanced by overland flow and infiltration.

Thus at this stage, overland flow is the major happening, which takes place over nearly the entire basin, and streamflow begins to bear some relation to the rate of precipitation. Subsurface flow also contributes to streamflow, and underground storage is replenished by ground water recharge (R) in some portion of the basin. The happenings taking place at this stage are shown in Fig. 7.82 (c).

(4) **After the end of rainfall.** When rain and overland flow ceases, the streamflow consists of only the baseflow and the channel storage. Evaporation takes place quite actively from soil moisture, depression and interception storages. Transpiration also starts from the vegetative cover. Water from depression storage also continues to infiltrate. Also the gravity water, still not drained up to the water-table and present in the zone of aeration, may continue its downward journey to join the water-table, and thus ground water recharge may continue. The water-table, consequently, may rise or fall, depending upon whether the downward percolation rate exceeds or not, the rate at which the ground water is contributing to the flow in the stream channel.

The conditions prevailing at this last stage of runoff cycle are shown in Fig. 7.82 (d).

7.38. Factors Affecting Runoff

The characteristics of the rainfall play an important part in determining the amount of consequent runoff. A light gentle rain may entirely be intercepted by vegetation, or it may be absorbed and stored in the soil. A sharp intense rainfall of short duration may result in large amount of runoff, because the rainfall rate greatly exceeds the infiltration rate.

The characteristics of the area on which water falls, also play a significant part in determining the quantity of runoff. The size of the watershed (drainage basin), the shape, the orientation, the topography, the geology, the surface vegetation, etc. play an important role in determining the quantity of runoff.

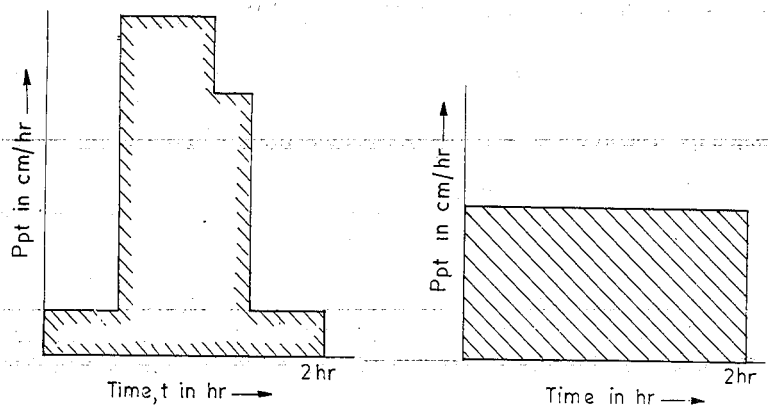
Thus, the various factors affecting the runoff, can be divided into two groups: (1) characteristics of precipitation; and (2) characteristics of drainage basin. Various factors falling under these two heads are discussed below:

(1) *CHARACTERISTICS OF PRECIPITATION*

(a) **Type of Precipitation.** Precipitation generally occurs either in the form of rain or snow. Runoff pattern or the hydrograph of runoff is considerably governed by this factor. Because, if the precipitation occurs in the form of rain, it will immediately produce a runoff (peak flow of short duration), while if the precipitation is in the form of snow, it will produce runoff at a slow and steady rate.

(b) **Rain Intensity.** Rain intensity has a lot of effect on the runoff. *If the intensity of rain increases, the runoff increases rapidly.* For example, if the intensity is increased four times, the runoff may increase nine times or so. For example, let there be a rain in progress for a time which is sufficient to make the infiltration capacity (f) a constant, say 0.5 cm/hr. Now, if the intensity of rain is 0.8 cm/hr, the runoff (strictly speaking excess rain) will occur at a rate of 0.3 cm/hr. Now let the intensity of rain be increased

to a value say 3.2 cm/hr. (four times) the resulting runoff rate will be equal to 2.7 cm/hr (nine times). Thus an intense rain of the type shown in Fig. 7.83 (*a*) will definitely produce much more runoff than a uniform rain of the type shown in Fig. 7.83 (*b*), provided the infiltration capacity remains the same throughout the storm period.



(*c*) Intense rain (*b*) Uniform rain

Fig. 7.83

Although the total amount of rain in Fig. 7.83 (*a*) and (*b*) is the same, still rain (*a*) will produce higher amount of runoff, while, rain (*b*) is likely to produce much less runoff.

(*c*) **Duration of Rainfall.** Duration of rainfall is important because of the fact that the infiltration capacity goes on reducing during the storm, till it attains a constant value. If the infiltration is less, the surface runoff will be more. Thus, in some cases, a longer duration rain may produce considerable runoff even when its intensity is mild. Further, if there is a rain extended over large periods of time, the watertable may rise quite high; say it may sometimes reach the ground, in which case, there will be no infiltration, and hence, there may be chances of serious flood hazards.

(*d*) **Areal Rainfall Distribution.** We have considered all the above factors in the light of the assumption that the rain is uniformly distributed over the entire basin; but in actual practice, it generally never happens. The rain may fall either on the whole basin or on a small part of it. *For small drainage basins, the peak flows are generally the result of intense rains falling over small areas. On the other hand for large drainage basins, the peak flows are the result of storms of lesser intensity but covering large areas.*

The runoff from a basin is thus very much dependent upon the areal distribution of rainfall. The areal rainfall distribution is generally expressed by the *areal distribution coefficient*, often called, *distribution coefficient.* The **distribution coefficient** for a given storm can be obtained by dividing the maximum rainfall at any point in the basin by the mean rainfall on the basin. Thus, if the distribution coefficient is more, it means that, rain is less uniformly distributed; and *hence for a given total rainfall and for other conditions remaining the same, the greater the distribution coefficient, the greater will be the peak runoff.* Moreover, even for the same distribution coefficient, the runoff may be more for the storm falling on the lower parts of the basin (near the outlet) and will be less for the storms falling on the upper parts of the basin (near the head waters).

(*e*) **Soil Moisture Deficiency.** The runoff depends upon the soil moisture present at the time of the rainfall. If a rain occurs after a long dry spell of time, the soil is dry

and it can absorb huge amounts of water, and thus even intense rain may fail to produce any appreciable runoff. But on the other hand, if the rain occurs after a long rainy season, the soil will be already wet and there may be very less infiltration, and even low intensity rain may cause peak flow and considerable stream rise, sometimes disastrous flood.

(*f*) **Direction of the Prevailing Storm.** If the direction of the storm is the same as the direction of the movement of the water in the drainage basin, water will remain in the basin for lesser period producing more runoff, as compared to the case when the storm is moving in the direction opposite to the water movement.

(*g*) **Other Climatic Conditions.** Various other climatic factors, such as temperature, wind, humidity, etc. affect the losses from the drainage basin, and, therefore, affect the runoff. If the losses are more, runoff will be less, and *vice-versa.*

(2) *CHARACTERISTICS OF THE DRAINAGE BASIN*

(*a*) **Size of the Basin.** If the area of the basin is large, the total flood flow will take more time to pass the outlet, thereby the base of the hydrograph of the flood flow will widen out, and consequently reducing the peak flow (because total volume of water passing is the same).

(*b*) **Shape of the Basin.** The shape of the drainage basin also governs the rate at which water enters the stream. The shape of a drainage basin is generally expressed by 'Form factor' and 'Compactness coefficient', as defined below:

$$\textbf{Form factor} = \frac{\text{Average width of the basin}}{\text{Axial length of the basin}} = \frac{B}{l} \qquad ...(7.93)$$

The axial length (l) is the distance from the outlet to the most remote point on the basin, and the average width (B) is obtained by dividing the area (A) by the axial length.

Therefore,

$$\textit{Form factor} = \frac{B}{l} = \frac{\frac{A}{l}}{l} = \frac{A}{l^2} \qquad ...(7.94)$$

Compactness coefficient is defined as :

$$\textit{Compactness coefficient} = \frac{\text{Perimeter of the basin}}{\text{Circumference of a circle whose area in equal to the area of the basin}} \qquad ...(7.95)$$

If A is the area of the basin and r_e is the radius of the equivalent circle, then $A = \pi r_e^2$.

$$\text{Circumference} = 2\pi r_e = 2\pi \cdot \sqrt{\frac{A}{\pi}} = 2 \cdot \sqrt{\pi A}$$

Therefore,

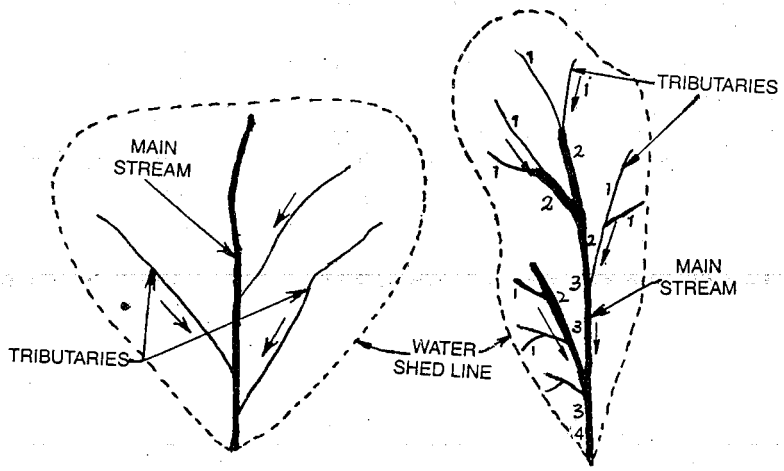
Compactness coefficient

$$= \frac{p}{2 \cdot \sqrt{\pi A}} \qquad ...[7.95(a)]$$

where p = perimeter of the basin

A = area of the basin.

There are **two types of catchments** in general, *i.e.* (*i*) fan shaped catchments (*ii*) fern leaf catchments, as shown in Fig. 7.84 (*a*) and (*b*). *Fan shaped catchments* give greater runoff because tributaries are nearly of the same size, and therefore time of flow is nearly the same and is smaller; whereas in *fern leaf catchments,* the time of concentration is more since the discharge is distributed over a long period, as is evident from the figures.



(*c*) Fan shaped catchment (*b*) Fern shaped catchment

Fig. 7.84. Types of catchments.

(*c*) **Elevation of the Watershed.** The elevation of the drainage basin governs the rainfall, its amount, and its type; and hence produces enough effect on the runoff.

The elevation of a water-shed is a variable factor from point to point. In order to determine the average elevation (Z) of the drainage basin, a contour map of the basin is taken, and the areas lying between successive contours are measured.

Z is then calculated as :

$$Z = \frac{A_1Z_1 + A_2Z_2 + A_3Z_3 \ldots\ldots A_nZ_n}{A} \qquad \ldots(7.96)$$

where A = area of the basin

$A_1, A_2, A_3 \ldots\ldots$ are the areas between successive contours

$Z_1, Z_2, Z_3 \ldots\ldots$ are the mean elevations between two successive contours.

(*d*) **The Type of Arrangement of Stream Channels.** If the **drainage network** (*i.e.* the arrangement of stream channels) of a catchment is *efficient*, water will flow rapidly, and will result in a higher peak, as the concentration time will be less. *Hence, the* more efficient is the drainage, the more flashy the stream flow will be, and *vice-versa*. The characteristics of the drainage net can be fairly described by the four factors: *i.e.* (*i*) order of streams; (*ii*) the length of tributaries; (*iii*) the stream density; and (*iv*) the drainage density, as discussed below:

(*i*) *Order of Streams.* All non-branching tributaries, regardless of whether they enter the main stream or its branches, are termed as *first order streams.* Streams which receive only non-branching tributaries are of the *second order.* Streams of the third order are formed by the junction of two streams of the second order. A third order channel receives flow not only from two second order channels that form it, but also direct overland flow and possibly from first order channels that run directly into it and possibly from other second order channels that might join it . Thus, a stream of any order has two or more tributaries of the next lower order. The ordering system continues in this fashion. Hence, the junction of two third order streams form a fourth order channel, and so on, as shown by in Fig. 7.84 (*b*), where order of streams are represented by Figs. 1, 2, 3 & 4 etc.

In accordance with this system, the order number of main stream indicates at once, the extent of bifurcation of its tributaries, and as a rule, is a direct indication of the size and extent of the drainage net.

(ii) Length of Tributaries. The length of the tributaries is an indication of the steepness of the drainage basin, as well as of the degree of drainage. Steep-well-drained areas generally have numerous small tributaries; whereas, in plains, where soils are deep and permeable, only relatively long tributaries (generally perennial streams) will be in existence. This factor, thus, gives the idea of the efficiency of the drainage net.

It is generally better to consider and compare the average length of the same type of tributaries, and especially of the first order of tributaries, than to compare the average length of all tributaries.

(iii) Stream Density. The **stream density** or *stream frequency* of a drainage basin may be expressed by relating the number of streams to the area drained. If N_s is the number of streams in the basin, and A is the total area, the stream density D_s, can then be expressed as:

$$D_s = \frac{N_s}{A} \text{ i.e. the number of streams per sq km.} \qquad ...(7.97)$$

In determining the total number of streams, only the perennial and intermittent streams are included.

This factor does not provide a true measure of drainage efficiency, because a basin having two smaller streams draining only a part of the basin, and another basin of the same size having two larger streams passing through the entire basin, will be indicated to be equally efficient by this factor, whereas it will not be so, the second being certainly more efficient than the first.

(iv) Drainage density. The **drainage density** is expressed as the length of stream per unit of area. Let D_d represents the drainage density, L the total length of the perennial and intermittent streams in the basin, and A the area; then

$$D_d = \frac{L}{A} \qquad ...(7.98)$$

Drainage density varies inversely with the length of the overland flow, and therefore, provides at least some indication of the drainage efficiency of the basin.

(e) **Other factors.** Besides these four important characteristics of the drainage basin, other factors, such as, the type of the soil in the catchment, the type of vegetative cover, the slope and orientation of the catchment, etc. are the various factors influencing the runoff. Some of these factors have already been described in the chapter of Infiltration. In fact, all those factors, on which Infiltration depends, can be quoted here also, because all those factors influence the streamflow.

From the above discussion, it is quite evident that the streamflow depends upon numerous factors (about 15 to 20 in number) and in order to express the streamflow by an equation is very difficult, because such an equation will involve numerous variables, which are all inter-dependent. Even with the most complicated mathematics, it seems difficult and rather impossible to evolve any generalised formula to evaluate the exact amount of runoff and the peak flow.

7.39. Certain Important Definitions connected with Runoff

Before we discuss the methods used for calculating runoff in our next chapter, let us define certain important terms, which are used in connection with runoff.

7.39.1. Time of Concentration (TOC). The time of concentration of a drainage basin is the time required by the water to reach the outlet from the most remote point of the drainage area. When a storm has been in progress for a time equal to the time of concentration, it is assumed that all the parts of the basin start contributing to the discharge at the outlet. For example, if a rain starts at 10 A.M., and at 10.30 A.M. the entire basin area just starts contributing water to the outlet, then this period of 30 minutes will be known as the time of concentration, generally denoted by T_c.

Note. The use of time of concentration for computing peak discharge and methods for its datermination are discussed in article 7.44.

7.39.2. Travel Time or Time of Flow. The excess rainfall produced in a basin flows overland on the basin surface, and then through the basin channels, before reaching the outlet point. The time taken by the water to reach the basin outlet, from the different points in the basin, is called the travel time or the flow time or the time of flow. This flow time will evidently be more for remote points of the basin and shall be less for the points which are nearer to the outlet. However, points can be earmarked on the basin, from where the flow will take equal time to reach the basin outlet. The line joining such points of equal time of flow is called an **isochrone**. Different isochromes can be drawn, representing different time of flow; and evidently the highest value of these isochromes will represent nothing but the time of concentration for the basin, since it represents the maximum flow time of the farthest points of the basin.

7.39.3. Period of Surface Runoff. It is the time taken by the surface runoff (S.R.O.) to pass the given section, after the surface runoff makes its first appearance.

7.39.4. Period of Rise. It is the time taken by the surface runoff to reach its maximum value from the time of its beginning.

COMPUTING RUNOFF FROM THE GIVEN RAINFALL

7.40. Fundamental Equation for Computation of Runoff (Volume or Depth on a Given Area)

The process of runoff has already been discussed in a previous article. It was stated there : that the rain is, first of all, intercepted as interception (P_i), then stored in depressions as depression storage (S_d), and then used in removing the soil-moisture deficiency. All this has to be accomplished before any streamflow or ground water accretion can start. The amount of precipitation required to fulfil these needs is generally termed as **initial basin recharge** and may be denoted by L. It includes P_i, S_d and rain absorbed by the soil moisture deficiency, which depends upon the prevailing soil moisture conditions at the time of rain. *Initial basin recharge is, therefore, the water withheld within the basin before the precipitation starts contributing to the steamflow or the ground water. After the initial recharge of the basin (L) is filled up, the water will infiltrate into the ground as ground water accretion (G), and excess water will flow as direct runoff (Q).*

The total precipitation (P) on a basin can, therefore, be easily represented by the equation

$$P = L + G + Q$$

or
$$Q = P - L - G \qquad ...(7.99)$$

i.e. Runoff = Precipitation – Initial Basin recharge – Ground water accretion

This is the fundamental equation for the computation of runoff. Various methods used for computing runoff are discussed below :

7.41. Computing Runoff Depth by Using Runoff Coefficient

The volume of runoff can be directly computed approximately, by using an equation of the form

$$Q = K \cdot P \qquad ...(7.100)$$

where Q = Runoff

P = Precipitation

K = a constant having a value les than 1 or at the most equal to 1. The value of K depends upon the imperviousness of the drainage area. Its value increases with the increase in imperviousness of the catchment area, and may approach unity (1.0) as the area becomes fully impervious.

Truly speaking, this equation cannot be rational, because the runoff not only depends upon the precipitation, but also depends upon the recharge of the basin. But the equation gives more and more reliable results, as the imperviousness of the drainage area increases and the value of K tends to approach unity.

This formula is frequently used in the design of storm water drains and small water control projects, especially for urban areas, where the percentage of the impervious area is quite high. This method of computing runoff should be avoided for rural areas and for analysis of major storms.

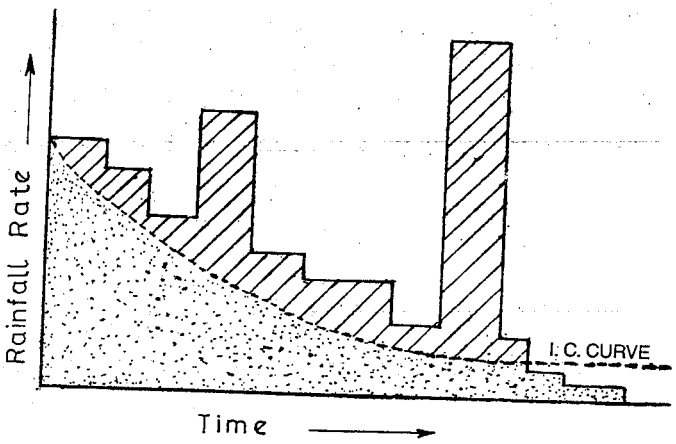
Various values of K, which are commonly used, are shown in Table 7.31.

Table 7.31. Values of Runoff Coefficient (K)

S. No.	*Type of area*	*Value of K*		
		Flat land 0 to 5% slope	*Rolling land 5% to 10% slope*	*Hilly land 10% to 30% slope*
(*a*)	Urban areas			
	30% area impervious (paved)	0.40	0.50	—
	50% area impervious (paved)	0.55	0.65	—
	70% area impervious (paved)	0.65	0.80	—
(*b*)	Single family residence in urban areas	0.3		
2.	*Cultivated Areas*			
	Open Sandy Loam	0.30	0.40	0.52
	Clay and Silt Loam	0.50	0.60	0.72
	Tight Clay	0.60	0.70	0.82
3.	*Pastures*			
	Open Sandy Loam	0.10	0.16	0.22
	Clay and Silt Loam	0.30	0.36	0.42
	Tight Clay	0.40	0.55	0.60
4.	*Wooded land or Forested Areas*			
	Open Sandy Loam	0.10	0.25	0.30
	Clay and Silt Loam	0.30	0.35	0.50
	Tight Clay	0.40	0.50	0.60

7.42. Computing Runoff by Using Infiltration Capacity Curve (I.C. Curve)

The infiltration capacity curve is a plot of the infiltration capacity against time. If the I.C. curve is superimposed on the rainfall hyetograph, the resultant amount will represent nothing but the runoff, as shown in Fig. 7.85.



Infiltration = Dotted lines
Runoff = Hatched
Fig. 7.85

The above method can be used very easily, if the rainfall rate never falls below the infiltration capacity rate. But natural rains of varying intensities, sometimes below and sometimes above the prevailing infiltration capacity, results in a distortion of the capacity time curve. It is generally assumed that the infiltration capacity at any time is determined by the mass infiltration, which has occurred upto that time. Thus, if a rain begins at low rates, and the rainfall during the first hour is two-third of the infiltration in one hour, the capacity rate at the end of the hour will be taken as the capacity that would have prevailed at $t = \frac{2}{3}$ hr and not at $t = 1$ hr.

7.43. Computing Runoff Depth by Using Infiltration Indices

The infiltration capacity curve already determined on test plots cannot be applied to large basins or heterogeneous areas. At any instant, in a large area, the infiltration capacity as well as the rainfall rate will vary from point to point. Moreover, subsurface flow (or interflow) will also be substantial. Since this water-flow is a part of infiltration, it will not normally be included in the runoff computed by using the I.C. curve determined on a test plot. Runoff volumes are generally computed for large areas by using *infiltration indices.*

W_{index} and ϕ_{index} are the two indices which are commonly used.

W_{index} is the average infiltration rate or the infiltration capacity averaged over the whole storm period, and is given by

$$W_{index} = \frac{F}{t_r} = \frac{P - Q}{t_r} \qquad ...(7.101)$$

where F = Total infiltration including initial basin recharge, called **potential infiltration**

P = Total precipitation

Q = Total runoff

t_r = Duration of rainfall in hr.

ϕ_{index} is defined as the average rate of loss such that the volume of rainfall in excess of that rate will be equal to the volume of direct runoff.

It can be defined, the other way round, as the rate of rainfall above which the rainfall volume equals the runoff volume.

ϕ_{index} can be represented graphically as shown in Fig. 7.86.

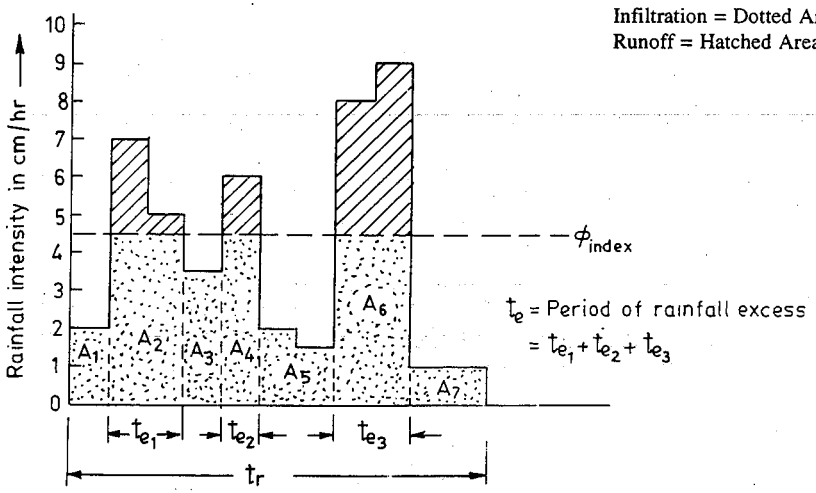


Fig. 7.86

It can be seen from this figure, that ϕ_{index} would be equal to :

$$\phi_{index} = \frac{\text{Total infiltration during period of rainfall excess}}{\text{Period of rainfall excess } (t_e)} \quad ...(7.102)$$

$$= \frac{A_2 + A_4 + A_6}{t_e} \quad ...[7.102\ (a)]$$

whereas, $W_{index} = \dfrac{A_1 + A_2 + A_3 + A_4 + A_5 + A_6 + A_7}{t_r}$ ...[7.101 (*a*)]

ϕ_{index} and W_{index} will evidently be equal for a uniform rain of the type shown in Fig. 7.87, but they would not be equal for a non-uniform rain of the type shown in Fig. 7.86.

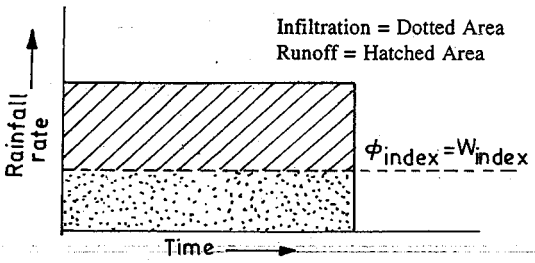


Fig. 7.87

However, for rains which are reasonably uniform or for heavy rains, these indices are found to be nearly equal. But in a usual case of moderate rain of non-uniform intensity, the ϕ_{index} will be somewhat higher than W_{index}. These indices will change with a change in interception, depression storage, and initial soil moisture. They will also change with the amount of precipitation. *These indices are not the actual infiltration rates, but merely measures of* **potential basin recharge,** which is sometimes, called *potential infiltration,* and includes infiltration as well as initial basin recharge.

The runoff coefficient K can be determined, if the value of W_{index} is known, by using the equation

$$K = \frac{p - W_{index}}{p} \qquad ...(7.103)$$

where p = rainfall rate.

Example 7.24. *The following are the rates of rainfall for successive 20 minutes period of a 140 minutes storm: 2.5, 2.5, 10.0, 7.5, 1.25, 1.25, 5.0 cm/hr. Taking the value of ϕ_{index} as 3.2 cm/hr, find out the net runoff in cm, the total rainfall and the value of W_{index}.*

Solution. The rain intensity pattern (*rainfall hyetograh*) can be plotted from the given rainfall rates, as shown in Fig. 7.88.

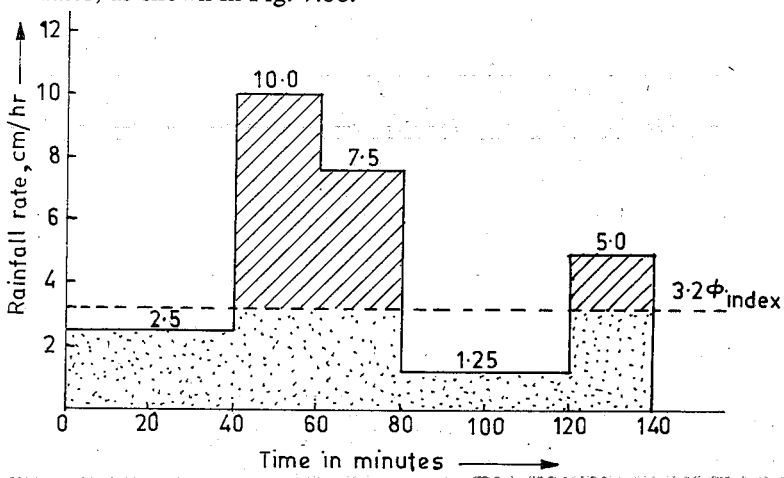


Fig. 7.88

ϕ_{index} line at a height of 3.2 cm/hr is superimposed. The hatched area is calculated, so as to obtain the value of runoff :

$$\text{Total Runoff } (Q) = (10 - 3.2)\frac{20}{60} + (7.5 - 3.2)\frac{20}{60} \times (5 - 3.2)\frac{20}{60}$$

$$= 6.8 \times \frac{20}{60} + 4.3 \times \frac{20}{60} + 1.8 \times \frac{20}{60}$$

$$= \frac{20}{60}(6.8 + 4.3 + 1.8) = \frac{20}{60} \times 12.9$$

$$= 4.3 \text{ cm.} \quad \textbf{Ans.}$$

Total precipitation (P)

$$= 2.5 \times \frac{40}{60} + 10 \times \frac{20}{60} + 7.5 \times \frac{20}{60} + 1.25 \times \frac{40}{60} + 5 \times \frac{20}{60}$$

$$= \frac{20}{60}(5.0 + 10 + 7.5 + 2.5 + 5) = \frac{20}{60} \times 30$$

$$= 10 \text{ cm} \quad \textbf{Ans.}$$

$$W_{index} = \frac{P-Q}{t_r \text{ in hr}} = \frac{10-4.3}{\left(\frac{140}{60}\right)} = \frac{5.7 \times 60}{140} = 2.44 \text{ cm/hr.} \quad \textbf{Ans.}$$

Example 7.25. *Tabulated below are data for a number of storms observed on a river. Compute the W_{index} for all storms. What would be the average error and the average percentage error in estimated runoff, if the average W_{index} was used to compute the runoff ? If the computed infiltration exceeds the rainfall, take the estimated runoff as zero.*

Storms No.	*Duration of rain in hours*	*Average rainfall in cm*	*Runoff in cm*
(1)	*(2)*	*(3)*	*(4)*
1	*12*	*2.82*	*1.32*
2	*48*	*2.98*	*1.02*
3	*24*	*4.55*	*2.46*
4	*72*	*14.22*	*7.42*
5	*18*	*2.87*	*0.43*
6	*24*	*3.91*	*0.48*
7	*36*	*8.10*	*1.93*
8	*12*	*4.40*	*1.67*
9	*30*	*5.31*	*1.98*
10	*18*	*6.98*	*3.15*

Solution. The given table is extended, so as to compute the value of W_{index} as shown in column (6) of table 7.32, which is self explanatory.

Table 7.32

Storm No.	*Duration of rain in hr = t_r*	*Av. rain-fall in cm P*	*Runoff in cm Q*	*P–Q cm*	*$\frac{P-Q}{t_r} = W_{index}$ cm/h*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1	12	2.82	1.32	1.50	0.1250
2	48	2.98	1.02	1.96	0.0408
3	24	4.55	2.46	2.09	0.0871
4	72	14.22	7.42	6.80	0.0944
5	18	2.87	0.43	2.44	0.1354
6	24	3.91	0.48	3.43	0.1430
7	36	8.10	1.93	6.17	0.1715
8	12	4.40	1.67	2.73	0.2275
9	30	5.31	1.98	3.33	0.1110
10	18	6.98	3.15	3.83	0.2128
			Σ = 21.86 cm		Σ = 1.3485

Average value of $W_{index} = \frac{1.3485}{10} = 0.13485 = \mathbf{0.13485 \text{ cm/hr.}}$ **Ans.**

If the average value of W_{index} *i.e.* 0.13485 cm/hr is used to evaluate runoff, then the computed values of runoff can be worked out as shown in column (6) Table 7.33. This table is also self explanatory.

Table 7.33

Storm No. (1)	*Duration t_r in hr* (2)	*Av. rainfall in cm P* (3)	*Av. $W_{index} \times t_r$ = 0.13485 t_r = Computed infiltration including basin recharge in cm* (4)	*Corrected infiltration (including basin recharge) in cm F* (5)	*Q = P–F = col. (3)–col. (5) cm* (6)
1	12	2.82	1.6182	1.6182	1.2018
2	48	2.98	6.4728	2.98*	0.0000
3	24	4.55	3.2364	3.2364	1.3136
4	72	14.22	9.7092	9.7092	4.5108
5	18	2.87	2.4273	2.4273	0.4427
6	24	3.91	3.2364	3.2364	0.6736
7	36	8.10	4.8546	4.8546	3.2454
8	12	4.40	1.6182	1.6182	2.7818
9	30	5.31	4.0455	4.0455	1.2645
10	18	6.98	2.4273	2.4273	4.5527
					Σ = 19.986 say 19.99 cm

From this table, the analysis is done as follows :

Average error introduced in runoff

$$= \frac{\Sigma Q \text{ (new computed)} - \Sigma Q \text{ (original given)}}{\text{No. of storms}}$$

$$= \frac{19.99 - 21.86}{10} = \mathbf{-0.187} \quad \textbf{Ans.}$$

Average percentage error

$$= \frac{19.99 - 21.86}{21.86} \times 100$$

$$= (-)\ \mathbf{8.55\%} \quad \textbf{Ans.}$$

The negative sign shows that by using the average value of W_{index}, the computed discharge has reduced.

Example 7.26. *A storm with a 15.0 cm precipitation produced a direct runoff of 8.7 cm. The time distribution of the storm is as follows :*

Time from start in hr	1	2	3	4	5	6	7	8
Incremental rainfall in each hr in cm	0.6	1.35	2.25	3.45	2.7	2.4	1.5	0.75

Estimate the ϕ_{index} of the storm. (*Civil Services, 1987*)

Solution. The hyetograph of rainfall is drawn in Fig. 7.89, by using the given values of rainfall of each hr, as they represent the rainfall intensity in cm/hr.

* Infiltration cannot be more than the rainfall.

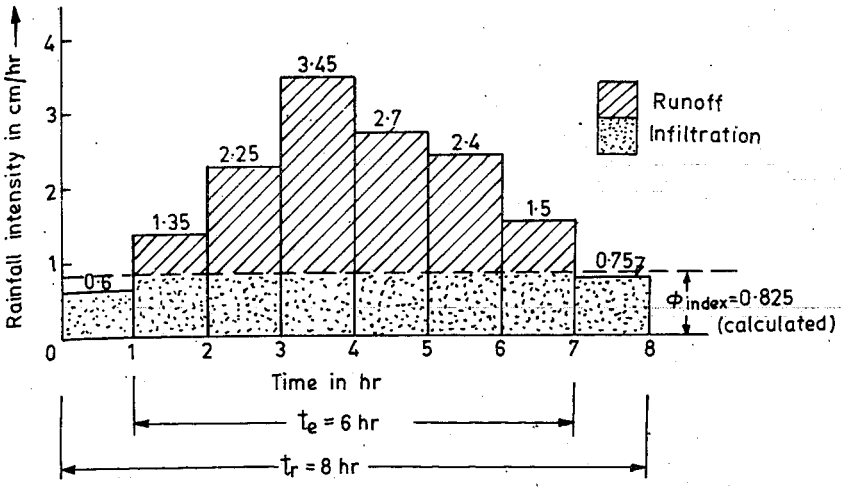


Fig. 7.89

Now, total precipitation $= P = 15.0$ cm

Total runoff $= Q = 8.7$ cm

$$W_{index} = \frac{P - Q}{t_r} = \frac{15 - 8.7}{8} \qquad [\because \quad t_r = \text{Rainfall duration} = 8 \text{ hr}]$$

$$= 0.7875 \text{ cm/hr}$$

Since ϕ_{index} has to be somewhat more than W_{index}, we can conclude that ϕ_{index} would be a little more than 0.7875 cm/hr. When this is so, evidently, the first hour rainfall and 8th hour rainfall would become ineffective in producing excess rain, because the rain intensity in those two hours would be less than ϕ_{index}.

In other words, the period (t_e) during which excess rain occurs, would be only $8 - 2 = 6$ hr. In that case, using equation (7.102), we have

$$\phi_{index} = \frac{\text{Total infiltration during period of excess rainfall}}{\text{Period of rainfall excess}}$$

$$= \frac{\text{[Total infiltration} - \text{Infiltration during the period when no excess rain occurs]}}{t_e}$$

$$= \frac{6.3 - 0.6 - 0.75}{6} \text{ cm/hr}$$

$$= \frac{6.30 - 1.35}{6} \text{ cm/hr}$$

$$= \frac{4.95}{6} = 0.825 \text{ cm/hr.}$$

This constant value of ϕ_{index} is marked as a straight line in Fig. 7.89, which confirms that there is no other period, except 1st and 8th hr, during which excess rain does not occur. Hence, the above calculated value of ϕ_{index} must be correct.

Hence, $\phi_{index} = 0.825$ cm/hr. **Ans.**

Check. Hatched area in Fig. 7.89 is runoff, which is calculated with ϕ_{index} of 0.825 cm/hr, as :

$$= (1.35 - 0.825)1 + (2.25 - 0.825)1 + (3.45 - 0.825)1 + (2.7 - 0.825)1 + (2.4 - 0.825)\,1 + (1.5 - 0.825)\,1 + (1.5 - 0.825)\,1 \text{ cm}$$
$$= 0.525 + 1.4252 + 0.625 + 1.875 + 1.575 + 0.675$$
$$= 8.7 \text{ cm (which is as given ; hence O.K.).}$$

Example 7.27. *A catchment area of 30 km² has one recording gauge. During a storm, the following mass curve of rainfall was recorded.*

Time from start of storm (h)	0	2	4	6	8	10	12	14
Accumulated rainfall (mm)	0	6	17	57	70	81	87	90

If the volume of the runoff due to the storm measured is 1.2×10^6 m³, estimate the ϕ_{index} of the catchment. (*AMIE, 1993*)

Solution.

Total Runoff (as volume) $= 1.2 \times 10^6 \text{ m}^3$

Catchment area $= 30 \text{ km}^2$

$$\therefore\ Q = \text{Total runoff depth} = \frac{\text{Total runoff volume}}{\text{Catchment area}} = \frac{1.2 \times 10^6 \text{ m}^3}{30 \times 10^6 \text{ m}^2} = 0.4 \text{ m} = 40 \text{ mm}$$

$$P = \text{Total rainfall} = 90 \text{ mm}$$

$$\text{Total Infiltration} = P - Q = 90 - 40 = 50 \text{ mm}$$

$$W_{index} = \frac{P - Q}{t_r} = \frac{90 - 40}{14} = \frac{50}{14} \text{ mm/h} = 3.57 \text{ mm/h}$$

Since ϕ_{index} has to be somewhat more than W_{index}, we can conclude that ϕ_{index} would be a little more than 3.57 mm/h; thereby causing infiltration to the ground in every 2 hr interval, some what more than $2 \times 3.57 \text{ mm} = 7.14 \text{ mm}$. The incremental rainfalls in different 2 hr intervals are now worked out in table 7.34.

Table 7.34

Time from start of storm in (h) (1)	*Accumulated rainfall in mm* (2)	*Incremental rainfall during each interval in mm* (3)
0	0	
2	6	6*
4	17	11
6	57	40
8	70	13
10	81	11
12	87	6*
14	90	3*
		$\Sigma = P = 90$ mm

* < 7.14 mm

It can be easily concluded from the values obtained in col. (3) above that no excess rainfall has occurred in 3 out of 7 intervals (each of 2 hours), when rainfall is lesser than the average infiltration. Hence, excess rain must have fallen only in 4 intervals *i.e.* in $4 \times 2h = 8$ hours, Hence, $t_e = 8$ hours. Using equation (7.102) we have

$$\phi_{index} = \frac{\text{Total infiltration during the period of excess rainfall}}{\text{Period of rainfall excess}}$$

$$= \frac{\text{Total infiltration} - \text{Infiltration during the period when no excess rain occurs}}{t_e}$$

$$= \frac{50 \text{ mm} - 6 \text{ mm} - 6\text{mm} - 3\text{mm}}{8\text{h}}$$

$$= \frac{35}{8} \text{ mm/h} = \mathbf{4.375 \text{ mm/h}} \quad \textbf{Ans.}$$

Example 7.28. *An isolated 3 hour storm occurred over an area of 120 ha as below:*

Partial area of catchment (ha)	ϕ_{index} *(cm/h)*	*Rainfall (cm)*		
		1st hour	*2nd hour*	*3rd hour*
36	*0.90*	*0.6*	*2.4*	*1.3*
18	*1.10*	*0.9*	*2.1*	*1.5*
66	*0.50*	*1.0*	*2.0*	*0.9*

What is the total rainfall on the catchment in this storm ? Estimate the runoff from the catchment. If the ϕ_{index} were to remain at the same value, what runoff would be produced by a uniform rainfall of 3.3 cm in 3 hours uniformly spread all over the catchment ? (*Civil Services, 1989*)

Solution. (*i*) Total rainfall (P) in 3 hr from 120 ha catchment

$= \Sigma$ Partial rainfalls, each in 3 hr

$= P_1 + P_2 + P_3$, where

partial rainfall volumes are:

$P_1 = 36\,(0.6 + 2.4 + 1.3)$ ha cm $= 154.8$ ha cm

$P_2 = 18\,(0.9 + 2.1 + 1.5)$ ha cm $= 81$ ha cm

$P_3 = 66\,(1.0 + 2.0 + 0.9)$ ha cm $= 257.4$ ha cm

$\therefore$ $P = (154.8 + 81 + 257.4)$ ha cm $= 493.2$ ha cm

Hence, total rainfall on 120 ha catchment (as volume)

$= 493.2$ ha cm

$\therefore$ Total rainfall as depth on this 120 ha catchment

$$= \frac{493.2}{120} \text{ cm} = \mathbf{4.11\ cm} \quad \textbf{Ans.}$$

Total runoff (volume) = Σ (Partial runoff volumes)

$$R_1 + R_2 + R_3, \text{ where}$$

$$R_1 = 36 \text{ ha } [0^* + (2.4 - 0.9)^* + (1.3 - 0.9)^* \text{ cm}]$$

$$= 36\ (1.5 + 0.4) = 68.4 \text{ ha cm}$$

$$R_2 = 18 \text{ ha } [0 + (2.1 - 1.1) + (1.5 - 1.1)]$$

$$= 18 \times 1.4 = 25.2 \text{ ha cm}$$

$$R_3 = 66 \text{ ha } [(1.0 - 0.5) + (2.0 - 0.5) + (0.9 - 0.5) \text{ cm}]$$

$$= 66\ [0.5 + 1.5 + 0.4] \text{ ha cm} = 158.4 \text{ ha cm}$$

$$R = 68.4 + 25.2 + 158.4 = 252.0 \text{ ha cm.}$$

Hence, total runoff from 120 ha catchment as volume

$$= \mathbf{252\ ha\ cm}$$

Total runoff (as cm depth) on 120 ha catchment

$$= \frac{252}{120} = \mathbf{2.1\ cm.} \quad \textbf{Ans.}$$

(*iii*) When uniform rain of 3.3 cm (in 3 hrs) falls on each partial catchment, then we have runoff from partial catchments, as :

$$R_1' = 36\ (3.3 - 3 \times 0.90) = 36 \times 0.6 \text{ ha cm}$$

$$R_2' = 18\ (3.3 - 3 \times 1.1) = 18 \times 0 = 0$$

$$R_3' = 66\ (3.3 - 3 \times 0.5) = 66 \times 1.8 \text{ ha cm}$$

Total Runoff $R' = 36 \times 0.6 + 0 + 66 \times 1.8$ ha cm = 140.4 ha cm.

Runoff (as depth) over 120 ha catchment

$$= \frac{140.4}{120} = \mathbf{1.17\ cm} \quad \textbf{Ans.}$$

7.44. Computing Peak Rate of Runoff (Runoff Intensity) By Rational Formula

In the design of hydraulic structures, sometimes the total volume of runoff is required, and sometimes the peak rate of runoff is required. This method helps us in evaluating the peak runoff rate**, and is discussed below:

If a rainfall is applied to an impervious surface at a constant rate, the resultant runoff from the surface would finally reach a rate equal to the rate of rainfall. In the beginning, only a certain amount of water will reach the outlet, but after sometime, the water will start reaching the outlet from the entire area, and in that case, the runoff rate would become equal to the rainfall rate. The time required to reach this *equilibrium condition* is known as the **time of concentration** (T_c). *For small impervious areas, we may assume that if a rain persists at a uniform rate for a period atleast as long as T_c, the peak rate of runoff will be equal to the rate of rainfall.* In other words, the peak runoff rate occurring for an area having coefficient of runoff K, as caused by a rainfall of intensity p, and continuing for a time more than T_c, will be given by ;

---

* hourly rainfall $\phi_{index} \rightarrow +$ ve values only.

** Various other methods for evaluating peak flows are given in article 7.50.

$$Q_p = K.p.A \quad \text{for } t \geq T_c \qquad ...(7.104)$$

This equation forms the basis of the rational formula, which on adoption of appropriate desired units for p in cm/h and A in hectares, states that:

$$Q_p = \left(\frac{1}{36}\right) K p_c \cdot A \qquad ...[7.105]$$

where Q_p = Peak rate of runoff in cumecs.

K = Coefficient of runoff ; values given in Table 9.1

A = Drainage area of the basin in hectares

p_c = Mean rainfall intensity in cm/hr for a duration equal to T_c and a given frequency of occurrence.

7.44.1. Relationship Between Intensity and Duration. Let P cm of rain has fallen in T hours during an individual storm. The mean rainfall intensity (p) will then be given by $p = \frac{P}{T}$ cm/hr. The intensity p' of the same storm during any other smaller time interval, say t hr., shall be more than the mean intensity over the whole period, because the intensity is not uniform throughout the storm, as shown in Fig. 7.90.

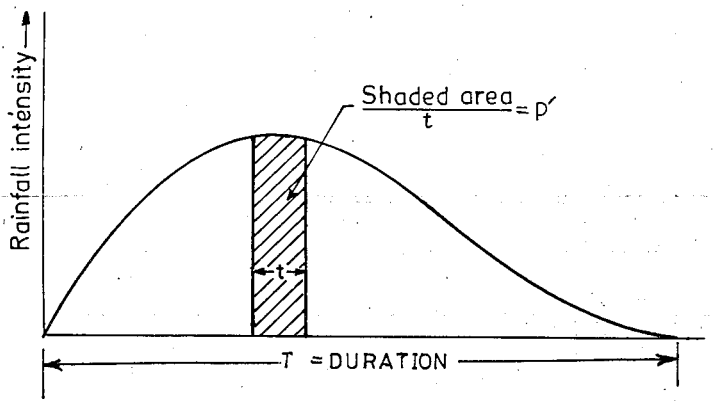


Fig. 7.90

Hence, the intensity of a storm is an inverse function of its duration. It can, therefore, be established that

$$\frac{p'}{p} = \frac{T+c}{t+c}, \text{ where } c \text{ is a constant}$$

From the rainfall statistics in general, the above value of c has been found to be equal to 1.0 when precipitation is measured in cm and time is measured in hours.

Thus $$\frac{p'}{p} = \frac{T+1}{t+1}$$

or $$p' = p\left(\frac{T+1}{t+1}\right) = \frac{P}{T}\left(\frac{T+1}{t+1}\right) \qquad ...(7.106)$$

where P is the total amount of rainfall and T is the duration of rainfall. Hence, by knowing the total amount of rainfall and its duration, the intensity of the storm (p') during any time (t) can be determined.

7.44.2. One Hour Rainfall. The intensity of the severest storm of a given frequency in a particular region, during a time interval of one hour, somewhere within the entire duration of the storm, is called One Hour Rainfall of the given frequency of that region, and is represented by p_0. Thus,

$$p_0 = \frac{P}{T}\left(\frac{T+1}{1+1}\right) = \frac{P}{2}\left(1+\frac{1}{T}\right)$$

The values of p_0 can be determined from P and T of various heavy and prolonged storms. The maximum value of p_0, thus found, can be accepted as 'the One Hour rainfall' of the region for a particular frequency for the design of hydraulic structures.

After deciding the frequency of the design storm, the value of p_0 *i.e.* 'one hour intensity' of that region can be taken from the tables or graphs and data supplied by Meteorological Department of the Government of India.* This value of point rainfall intensity is multiplied by the **areal distribution factor,** so as to obtain p_0. The areal dispersion factor may vary from 1.0 for small areas to 0.80 for very large areas of the order of 20,000 hectares or so. The p_0 can then be reduced to p_c by using eqn. (7.106), as

or $$\frac{p_c}{p_0} = \frac{1+1}{T_c+1} = \frac{2}{1+T_c}$$

or $$p_c = p_0\left(\frac{2}{1+T_c}\right) \qquad ...(7.107)$$

where T_c is the time of concentration of the given catchment.

The discharge, can then be calculated by using Rational formula as

$$Q = \left(\frac{1}{36}\right) K \cdot p_0 \left(\frac{2}{1+T_c}\right) \cdot A \text{ m}^3/\text{s} \qquad ...(7.108)$$

where A is in hectares and p_0 in cm/hr.

7.44.3. Estimating the Concentration Time of a Catchment. The Rational formula is generally used for the design of storm water drains and other structures handling runoff from small areas.

For small plots having no defined flow channels, and from which runoff occurs as laminar overland flow, Izzard has suggested the following formula for evaluating the time of concentration in minutes.

Time of concentration for plots having no channels, called *overland flowtime* or *Inlet time* (T_0), is given by :

$$T_0 = \frac{111 \cdot b \cdot (L_0)^{1/3}}{(Kp)^{2/3}} \text{ minutes} \qquad ...(7.109)$$

* Values of one hour rainfall for different frequencies (such as 2 years, 5 years and 50 years) at various places in India, are shown in the enclosed plates, numbered as 7.1, 7.2, and 7.3 respectively.

where L_0 = Length of overland flow in metres.

p = Rainfall intensity in cm/hr.

K = Runoff coefficient

b = a coefficient, whose value is given by

$$b = \frac{0.000275p + C_r}{(S_0)^{1/3}} \qquad ...(7.110)$$

where S_0 = Slope of the surface

C_r = Retardance coefficient ; values given in Table 7.35

These equations are applicable only when the product $p \cdot L_0$ is less than about 387.

Table 7.35. Values of Retardance Coefficient (C_r)

S. No.	*Type of Surface*	*Value of C_r*
1.	Smooth asphalt surface	0.007
2.	Concrete pavement	0.012
3.	Tar and gravel pavement	0.017
4.	Closely clipped soil	0.046
5.	Dense blue grass turf	0.060

However, for the design of hydraulic structures, the following simple formula called **Kirpich formula,** can be used for evaluating the inlet time or *overland flow time.*

$$T_0 = \left(0.885 \frac{L_0^3}{H}\right)^{0.385} \qquad ...(7.111)$$

where, T_0 is the overland flow in hours.

L = The distance from the farthest (*i.e.* critical) point to the outlet at hydraulic structure, such as culvert etc., in kilometres.

H = Total fall in level from the critical point to the outlet, such as culvert, etc., in metres.

L and H can be found from the survey plan of the area, and T_0 can be easily calculated.

Time of concentration for a small drainage basin having flow channels in it, would be equal to the longest combination of overland flow time called *Inlet Time* (T_0) and *Channel flow time* (T_f) which exists anywhere in the basin. Channel flow time T_f is generally taken as the length of the longest channel divided by the average flow velocity in the channel at about bankful stage.

Hence $T_c = T_0 + T_f$ ...(7.112)

Valley Storage and Limitations of Rational Formula. As the rate of overland flow and the rate of channel flow increases, the depth of flow also increases. As the depth of flow increases, a certain quantity of water is stored temporarily until the runoff rate again decreases. This water which is stored temporarily within the channel of the basin is called *channel storage or* **valley storage.** Equilibrium conditions will occur only when a sufficient amount of rain has occurred to supply this *valley storage.*

This is comparable to a barrel fed by a uniform inflow and emptied through an orifice at the outlet. Equilibrium conditions will be obtained when the inflow in the barrel becomes equal to the outflow from the barrel. This will happen only when the depth of the barrel is sufficient to cause discharge through the orifice at a rate equal to the inflow. If the diameter of the barrel is increased, more depth will be required for the equilibrium condition, and hence, time of equilibrium will be more. Thus, if the drainage area increases, the true value of T_c becomes much larger than the transit time obtained by dividing channel length by flow velocity. But with the increase of time, the likelihood of a continuous rain at a uniform rate decreases. Both these factors affect the Rational formula, and hence, Rational formula no longer remains rational and the term 'Rational' becomes misleading because for longer areas, the results obtained by this formula will be erroneous. This formula should therefore be used with caution for areas more than about 1000 hectares and is not at all applicable for areas more than about 5000 hectares.

Example 7.29. *An area of 5 hectares in a single family residence district has an average length of overland flow of 40 metres, average slope of the lots is 0.003, and the design rainfall is given by $p = 64/t^{0.5}$, where p is in cm/h and t is in minutes. Ignoring the intensity term in the retardance coefficient equation (i.e. equation 7.110) and using $C_r = 0.05$; find the time of concentration of the overland flow from this area. Assuming gutter flow time to add 10 min., find the peak rate of runoff to be expected.*

Solution. Equation (7.109) states

$$T_0 = \frac{111\, b \cdot (L_0)^{1/3}}{(K \cdot p)^{2/3}} \text{ minutes}$$

where,
$$b = \frac{0.000275\, p + C_r}{(S_0)^{1/3}} \text{ (from Eq. 7.110)}$$

Ignoring p term from the value of b, we get

$$b = \frac{C_r}{(S_0)^{1/3}}$$

$$= \frac{0.05}{(0.003)^{1/3}} = \frac{0.05}{(0.003)^{0.33}} = \frac{0.05}{0.144} = \mathbf{0.347}$$

Now, from Eq. (7.109)

$$T_0 = \frac{111\, b \cdot (L_0)^{1/3}}{(K \cdot p)^{2/3}}$$

From Table 7.31, the value of K for a single house is 0.3

and
$$p = \frac{64}{(T_0)^{0.5}} \text{ (given)}$$

$$\therefore \quad K \cdot p = 0.3 \times \frac{64}{(T_0)^{0.5}} = \frac{19.2}{(T_0)^{0.5}}$$

or
$$(K \cdot p)^{2/3} = \left[\frac{19.2}{(T_0)^{0.5}}\right]^{2/3} = \frac{(19.2)^{0.667}}{(T_0)^{0.333}} = \frac{7.18}{(T_0)^{0.333}}$$

Putting the above values of b and $(K \cdot p)^{2/3}$ in equation (7.109), we get

$$T_0 = \frac{111 \times 0.347 \times (40)^{1/3}}{\frac{7.18}{(T_0)^{0.333}}} \text{ minutes}$$

or $$\frac{T_0}{T_0^{0.333}} = \frac{111 \times 0.347 \times (3.42)}{7.18}$$

or $$T_0^{0.667} = 18.35$$

or $$T_0 = (18.35)^{1/0.667} = (18.35)^{1.5} = 79 \text{ minutes.}$$

Hence, the time of concentration of the overland flow from this area

$= 79$ minutes. **Ans.**

Gutter flow time $= T_f = 10$ min.

T_0 obtained above is the time of concentration without channel flow time (T_f).

$\therefore$ Total time of concentration (T_c)

$= 79 + 10 = 89$ minutes.

Now $$p = \frac{64}{t^{0.5}}$$

$\therefore$ $$p_c = \frac{64}{(T_c)^{0.5}} = \frac{64}{(89)^{0.5}} = = \mathbf{6.79\ cm/hr.}$$

and $$Q_{max} = \frac{1}{36} K \cdot p_c \cdot A$$

$$= \frac{1}{36} \times 0.3 \times 6.79 \times 5 \text{ cumecs}$$

$= 0.283$ cumecs. **Ans.**

Example 7.30. *The total catchment area of 4000 hectares of a stream upto a gauging site is subdivided into four parts A, B, C and D as shown below, such that the time required for the water falling on the ground to flow to point P, are :*

For any point on division line $EF = 1$ *hr*
For any point on division line $GH = 2$ *hr.*
For any point on division line $JK = 3$ *hr.*
For any point on division line $LM = 4$ *hr.*

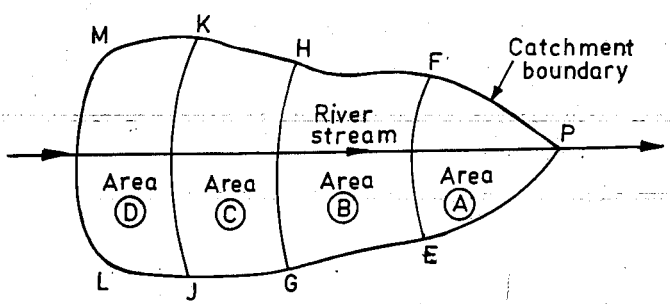


Fig. 7.91

Over this catchment area, a storm occurs for 4 hours, uniformly falling over the entire area. The total rainfall is 36 mm. Determine the peak flow and the hydrograph of resulting runoff at P, if the runoff factor is constant throughout the storm for the entire area as equal to 0.8. Explain the theory which helps you in computation of this hydrograph.

Solution. Theory : When a rain falls uniformly over the area A for 1 hour, the hydrograph of runoff produced will be a triangle starting from 0 hr, reaching its peak at 1 hr, and again falling to zero value at 2 hr, as shown in Fig. 7.92 (*a*), because the entire area A starts contributing to discharge at P after 1 hour.

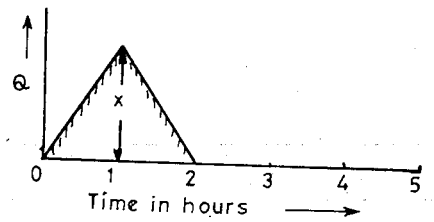


Fig. 7.92 (*a*) Hydrograph caused by the Ist hr. rain from an area having time of concentration of 1 hr.

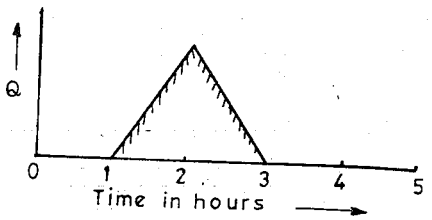


Fig. 7.92 (*b*) Hydrogaph caused by 2nd hr. rain from an area having time of concentration of 1 hr.

If rain further continues beyond 1 hr, for another hour, then the runoff caused by that additional 2nd hour rain, would also be a triangle starting from 1 hr, and reaching its peak at 2 hr, and to zero value at 3 hr, as shown in Fig. 7.92 (*b*).

The total discharge hydrograph caused by this composite 2 hr rainfall would be obtained by addition of these two hydrographs, as given in Fig. 7.92 (*c*).

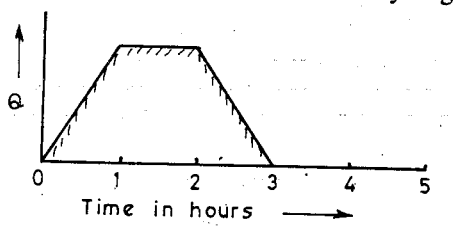


Fig. 7.92 (*c*) Hydrograph caused by continuous 2 hour uniform rain from an area having time of concentration equal to 1 hr.

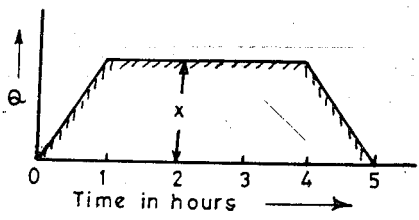


Fig. 7.92 (*d*) Hydrogaph caused by continuous 4 hour uniform rain from an area having time of concentration equal to 1 hr.

Similarly, the hydrograph caused by a rainfall of 4 hr over area A, will be as shown in Fig. 7.92 (*d*).

Similarly, when such a rain falls on the area B, the hydrograph of runoff would be made up of similar triangle, but with the difference that its rising limb will start after 1 hr, and its peak will reach at time equal to 2 hr (because time lag for the area B, *i.e.* time of concentration, is 2 hr), as shown in Fig. 7.93 (*a*). Hydrographs for other conditions are also shown in Figs. 7.93 (*b*) to (*d*).

The hydrograph caused by 4 hr rain over area A as well as over area B, can be obtained by adding the separate hydrographs of areas A and B, *i.e.* by adding the hydrographs of Fig. 7.92 (*d*) and 7.93 (*d*), as shown in Fig. 7.94.

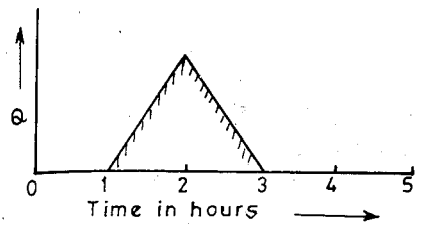


Fig.7.93 (*a*) Hydrograph caused by Ist hr. rain over area *B* having time of concentration of 2 hr.

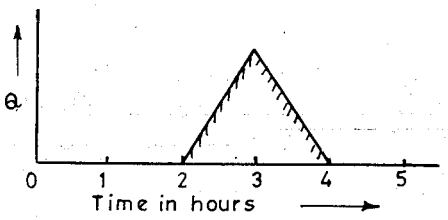


Fig. 7.93 (*b*) Hydrograph caused by 2nd hr. rain area *B* having time of concentration of 2 hr.

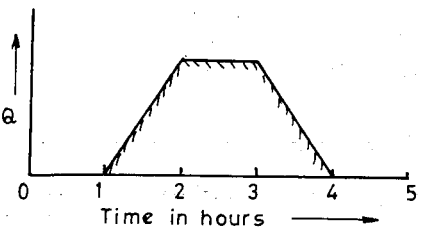


Fig.7.93 (*d*) Hydrograph caused by composite 2 hr. uniform rain over area *B*, having time of concentration of 2 hr.

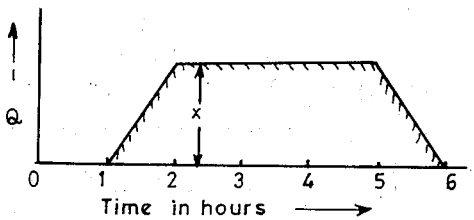


Fig. 7.93 (*d*) Hydrograph caused by composite 4 hr. rainfall on area *B* having time of concentration of 2 hr.

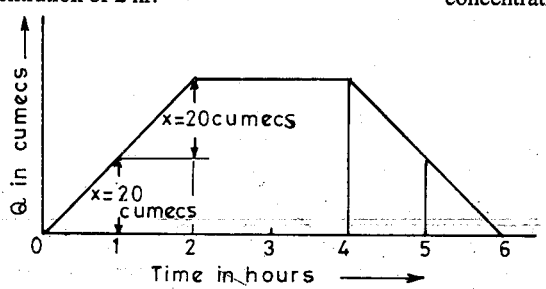


Fig. 7.94. Hydrograph of 4 hour rain over area *A* and *B* together.

The above additions of hydrographs for the data given in Example 7.30 for all the 4 areas *A*, *B*, *C*, and *D* can be easily done analytically in a tabular form, as shown in table 7.36 for the following given data :

Total 4 hr uniform rain = 36 mm

Rain in each hr = 9 mm

Runoff coefficient = 0.8

Runoff caused by each hr rain

$$= 9 \times 0.8 = 7.2 \text{ mm}$$

Areas $A = B = C = D = \dfrac{4000}{4} = 1000$ hectares

$$= 1000 \times 10^4 \text{ m}^2 = 10^7 \text{ m}^2$$

Area of each Δ = runoff caused by each 1 hr rain due to each part area.

$$\therefore \quad \frac{1}{2} \cdot (2 \text{ hr}) \cdot x = 7.2 \text{ mm}$$

$$\therefore \quad x = 7.2 \text{ mm/hr}$$

$$= \frac{7.2}{10 \times 100} \times \frac{10^7}{3600} \text{ m}^3/\text{sec} = 20 \text{ m}^3/\text{sec}.$$

Table 7.36. Discharges caused during 4 hr rain falling over areas *A*, *B*, *C* and *D*

Time at the end of hr	*Discharge caused by area A cumecs*	*Discharge caused by area B cumecs*	*Discharge caused by area C cumecs*	*Discharge caused by area D cumecs*	*Total discharge caused by entire area cumecs*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0				0
1	20	0			20
2	20	20	0		40
3	20	20	20	0	60
4	20	20	20	20	80
5	0	20	20	20	60
6		0	20	20	40
7			0	20	20
8				0	0

The total discharge caused at various hrs, as worked out in column (6) is plotted to obtain the runoff hydrograph, as shown in Fig. 7.95.

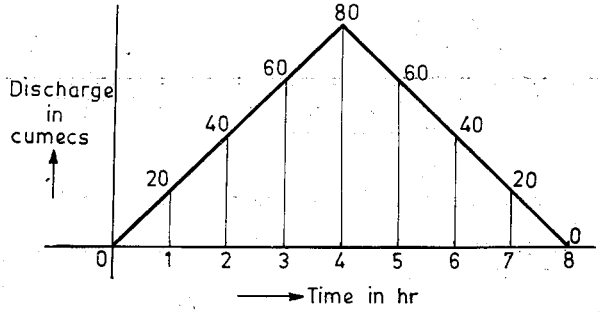


Fig. 7.95. Hydrograph of 4 hr. rain over entire area.

The peak discharge would be 80 cumecs. **Ans.**

Example 7.31. *In estimating the peak discharge of a river at X, the catchment area was divided into four parts A, B, C and D. The time of concentration and the area for different parts are as follows :*

Part	*Time of concentration*	*Area in sq m*
A	*1 hr*	600×10^4
B	*2 hr*	750×10^4
C	*3 hr*	1000×10^4
D	*4 hr*	1200×10^4

Records for a rain storm lasting for 4 hr, as observed, and the runoff factor during different hours are as follows :

Time in hr from – to	*Rainfall in mm*	*Runoff factor*
0—1	*25.0*	*0.50*
1—2	*50.0*	*0.70*
2—3	*50.0*	*0.80*
3—4	*23.5*	*0.85*

Calculate the maximum flow to be expected at X in cumecs assuming a constant base flow of 42.5 cumecs. *(U.P.S.C., Engg. Services, 1985)*

Solution. This question involves the method, which has been explained in the previous example with the difference that here, the rainfall and runoff coefficients are also varying during the respective 4 hrs, and also the areas are different. Hence, the peak runoff values, caused due to different 1 hr rains, would be different.

The computations are carried out in Tables 7.37 and 7.38, which are self explanatory.

Table 7.37

Time from start (1)	*Hourly rainfall in mm* (2)	*Runoff coefficient* (3)	*Hourly runoff depths in mm col. (2)× col. (3)* (4)
0			0
1	25	0.5	12.5
2	50	0.7	35.0
3	50	0.8	40.0
4	23.5	0.85	20.0

The values of discharges caused by different areas at different hours, are finally tabulated and added as shown in table 7.39, including the base flow. The table is otherwise self explanatory. It can be seen from col. (7) of this table that the peak discharge is equal to 311.92 cumecs and occurs at 5 hours from start. The hydrograph of runoff can also be plotted by using the values of cols (1) and (7) of Table 7.39, as shown in Fig. 7.96.

Table 7.38

Rain intervals	*Height of Triangular Hydrograph Caused by*			
	Area A (600×10^4 m^2) *in* m^3/sec	*Area B* (750×10^4 m^3) *in* m^3/sec	*Area C* (1000×10^4 m^2) *in* m^3/sec	*Area D* (1200×10^4 m^2) *in* m^3/sec
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1st hr rain (causing 12.5 mm runoff)	$\frac{1}{2} \cdot x \cdot 2$ hr = 12.5 mm $= \frac{12.5}{10 \times 100} \times 600 \times 10^4$ m^3 $\therefore x = \frac{12.5 \times 6 \times 10^6}{10^3 \times 3600}$ **cumecs** **= 20.83 m^3/s** at t = 1 hr	$\frac{750}{600} \times 20.83$ **= 26.03 cumecs** at t = 2 hr	$\frac{1000}{600} \times 20.83$= **34.72 cumecs** at t = 3 hr	$\frac{1200}{600} \times 20.83$ **= 41.66 cumecs** at t = 4 hr
2nd hr rain (causing 35 mm runoff)	$\frac{1}{2} \cdot x \cdot 2$ hr = 35 mm $= \frac{35}{10 \times 100} \times 600 \times 10^4$ m^3 $\therefore x = \frac{35}{10^3} \times \frac{6 \times 10^6}{3600}$ m^3/sec **= 58.33 cumecs** at t = 2 hr	$\frac{750}{600} \times 58.33$ **= 72.91 cumecs** at t = 3 hr	$\frac{1000}{600} \times 58.33$= **97.22 cumecs** at t = 4 hr	$\frac{1200}{600} \times 58.33$ **= 116.66 cumecs** at t = 4 hr
3rd hr rain (causing 40 mm runoff)	$\frac{1}{2} \cdot x \cdot 2$ hr = 40 mm $= \frac{40}{10 \times 100} \times 600 \times 10^4$ m^3 $\therefore x = \frac{40}{10^3} \times \frac{6 \times 10^6}{3600}$ m^3/sec **= 66.66 cumecs** at t = 3 hr,	$\frac{750}{600} \times 66.66$ **= 83.33 cumecs** at t = 4 hr	$\frac{1000}{600} \times 66.66$ **= 111.1 cumecs** at t = 5 hr	$\frac{1200}{600} \times 66.66$ **= 133.32 cumecs** at t = 6 hr
4th hr rain (causing 20 mm runoff)	$\frac{1}{2} \cdot x \cdot 2$ hr = 20 mm $= \frac{20}{10 \times 100} \times 600 \times 10^4$ $\therefore x = \frac{20}{10^3} \times \frac{600 \times 10^4}{3600}$ m^3/sec **= 33.33 cumecs** at t = 4 hr	$\frac{750}{600} \times 33.33$ **= 41.66 cumecs** at t = 5 hr	$\frac{1000}{600} \times 33.33$ **= 55.55 cumecs** at t = 6 hr	$\frac{1200}{600} \times 33.33$ **= 66.66 cumecs** at t = 7 hr

Table 7.39

Time in hr.	*Discharge caused by area A in cumecs*	*Discharge caused by area B in cumecs*	*Discharge caused by area C in cumecs*	*Discharge caused by area D in cumecs*	*Discharge caused by base flow in cumecs*	*Total Discharge = (2) + (3) + (4) + (5) + (6) in cumecs*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
0	0				42.50	42.5
1	20.83	0			42.50	63.33
2	58.33	26.03	0		42.50	126.86
3	66.66	72.91	34.72	0	42.50	216.79
4	33.33	83.33	97.22	41.66	42.50	298.04
5	0	41.66	111.10	116.66	42.50	**311.92**
6		0	55.55	133.32	42.50	231.37
7				66.66	42.50	109.16
8				0	42.50	42.5

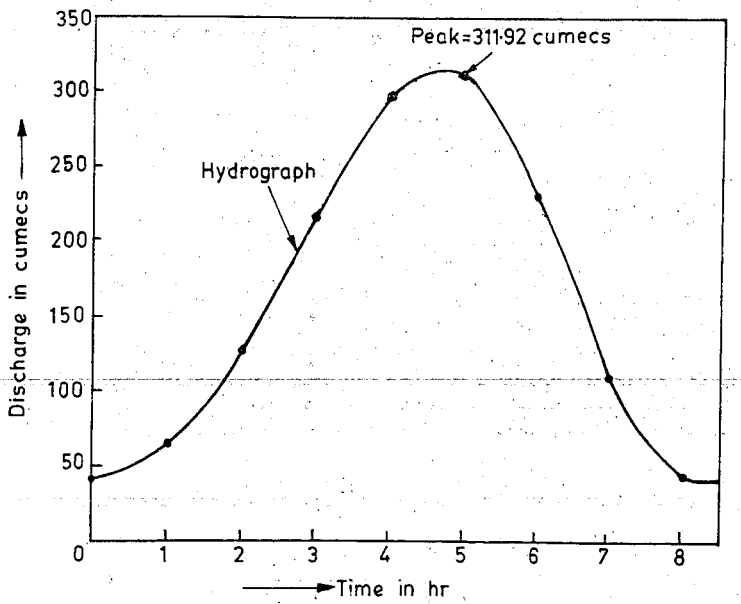


Fig. 7.96

7.45. Computing Runoff Hydrograph by Using Unit Hydrograph Theory

So far, we have discussed the methods, which help us in computing single value of runoff, resulting from a given rain value, However, unit hydrograph theory helps us in computing the entire hydrograph of surface runoff, resulting from the given rainfall hyetograph. This important theory is discussed below in details :

7.45.1. Unit Hydrograph Theory. The concept of a unit hydrograph (U.H.), initially called a *unit graph*, was propounded by *Mr. L.K. Sherman* in 1932, and provides a very versatile tool for determining the *hydrograph of surface runoff** resulting from

* also called *Direct Runoff Hydrograph (DRH).*

any given amount of *rainfall excess**. The method involves the preparation of standard unit hydrographs caused by rainfalls of specified durations, such as 4 hours, 6 hours, 12 hours, etc. This standard T-hour unit hydrograph containing one cm of excess rain is used to work out the hydrograph caused by any other rain of the same duration and containing any other amount of excess rain.

A **T-hour unit hydrograph** *is thus defined as the hydrograph of runoff produced by an intense excess rainfall of 1 cm*** *occurring uniformly over the entire drainage basin and at a uniform rate for the short specified duration of T hour.* Since the rainfall excess is nothing but surface runoff, the volume of water contained in the unit hydrograph shall be equal to 1 cm depth over the basin area.

The specified duration of T hour was termed by Mr. Sherman as the *unit duration*, and the storm of this duration was called the *unit storm.*

The word "unit" in the unit graph was, thus used by its originator to stand for the unit of time, yet however, since its inception, it has often been interpreted to stand for the unit depth (1 cm or 1 inch) of excess rainfall contained in the U.H.

The application of a unit hydrograph of 1 cm excess rain for the computation of the direct runoff hydrograph resulting from an excess rain of D cm, or *vice versa*, is based upon the *following assumptions :*

(*i*) *Time Invariance.* The runoff produced from a given drainage basin due to a given effective rainfall, shall always be the same irrespective of the time of its occurrence. In other words, the runoff response of a basin to a given effective rainfall is assumed to be time invariant.

(*ii*) *Linear Response.* The runoff response of a drainage basin to the excess rainfall is assumed to be linear. Linear response means that if an input $x_1(t)$ causes an output $y_1(t)$, and an input $x_2(t)$ causes an output $y_2(t)$, then an input $x_1(t) + x_2(t)$ will cause an output $y_1(t) + y_2(t)$. In other words, an input equal to $D \cdot x_1(t)$ will cause an output equal to $D \cdot y_1(t)$. It therefore follows that if the excess rainfall of D *cm* occurs in a duration of T *hour*, then the resulting runoff hydrograph will have its ordinates equal to D times the ordinates of unit hydrograph. Since the area of the resulting direct runoff hydrograph shall increase by the ratio D, the base of this hydrograph shall be the same as that of the unit hydrograph. The hydrograph of any other storm of specified duration (T hour) will thus, have the same shape as that of the unit hydrograph of the same duration (T hour), but with ordinates of flow in proportion to their runoff volumes.

Storms of different intensities producing different volumes of excess rain, but of a particular duration, will thus, have the same shape of their direct runoff hydrographs. Different unit hydrographs for different durations can be prepared, and a particular unit hydrograph when once prepared for a particular duration, can be utilised for evaluating the runoff hydrograph of any other storm of the same duration by multiplying the ordinates of unit hydrograph by D. The assumption of linear response in a unit hydrograph thus enables the method of super-position to be used to derive the direct runoff hydrographs. Accordingly, if two rainfall excesses each of T hour duration occur consecutively, then their combined effect can be obtained by superposing the respective runoff hydrographs, with due care being taken to account for the time lag between the two sequence of events.

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* also called *effective rainfall.*

** 1 inch in FPS units.

Limitations of Unit Hydrograph Theory. The **basic assumptions** made in defining a U.H. were that : (*i*) *the excess rain should occur uniformly over the entire basin*; and (*ii*) *that its intensity should be constant during the entire duration.* In actual practice, however, these two conditions are never strictly satisfied, since storms do have non-uniform areal distribution, and their intensities also vary during the specified duration.

Even under *non-uniform areal distribution of rainfall,* unit hydrographs can be used, if the variations in areal extent are consistent within the different storms, which is reasonably so for intense rains falling in smaller catchments. This, however, imposes a limit on the size of the catchment, since for very large basins, the centre of the storm can vary from storm to storm, and each can give different surface runoff hydrograph, under otherwise identical situations. *Unit hydrographs therefore, cannot give reliable results for basins exceeding about 5000 sq. km. or so.* Such large basins can however be studied by U.H. theory by dividing them into smaller sub-basins. The direct runoff hydrographs for all such sub-basins can then be developed by using their respective unit hydrographs, which can be routed through the respective flow channels to obtain the composite hydrograph at the basin outlet.

Unit hydrographs are also generally not developed and used for very very small basins, having areas lesser than say 2 sq. km. or so, because for such small basins, several other factors do affect the rainfall-runoff relationship, making the unit hydrographs to give inaccurate results.

Variations in rainfall intensity during the specified duration can also affect the accuracy of unit hydrographs, which can be minimised by choosing a smaller duration, which according to Mr. Sherman was not to exceed the time of concentration. Other investigators limit this duration to the *period of rise,* which is more easily known than the time of concentration, and being a little shorter than the time of concentration. Hence, for basins having very large variations in rainfall intensity, the specified duration must not exceed about $\frac{1}{2}$ of the period of rise. *Moreover, since the valley storage tends to neutralise the effects of small variations in rainfall intensity, somewhat longer durations may be chosen for basins with larger valley storages.* It is a good rule to select a duration of 12 hrs for basins over 2500 sq. km. ; 12, 8 and 6 hr for basins between 2500 to 250 sq. km.; and 4 and 2 hr for catchments of 250 to 50 sq. km. Where rainfall records of less than daily rainfall are not at all available, and the basin area is more than 1250 sq. km., even 1 day *i.e.* 24 hour period may be adopted as the specified duration for deriving the unit hydrograph.

Uses of Unit Hydrograph Theory. Unit hydrographs as stated earlier, have proved to be excellent tools in analysing the hydrology of a catchment, since they establish a relationship between Excess Rain Hyetograph (ERH) and Direct Runoff Hydrograph (DRH). *They are chiefly used for :*

(i) developing flood hydrographs for extreme rainfall magnitudes, for design of hydraulic structures ;

(ii) extending flood flow records by using the available rainfall records ; and

(iii) developing flood forecasting and warning system based on rainfall data.

7.45.2. Preparing a Unit Hydrograph from an Isolated Storm. First of all, the runoff rates (in cm/hr) resulting from the given storm are plotted against time (in hours), so as to obtain *direct run off hydrograph for the given storm.* The total area

(D) contained within this hydrograph is calculated by a planimeter or by Simpson's rule, etc. This will represent nothing but the volume of runoff in cm depth of water on the basin.

The ordinates of this hydrograph ABC (Fig. 7.97) are multiplied by $\frac{1 \text{ cm}}{D \text{ cm}}$, so as to obtain the ordinates of a unit hydrograph $AB'C$. For example, suppose the area ABC is 3 cm depth of water, then ordinates of unit hydrograph can be obtained by multiplying the ordinates of storm hydrograph by $\frac{1}{3}$.

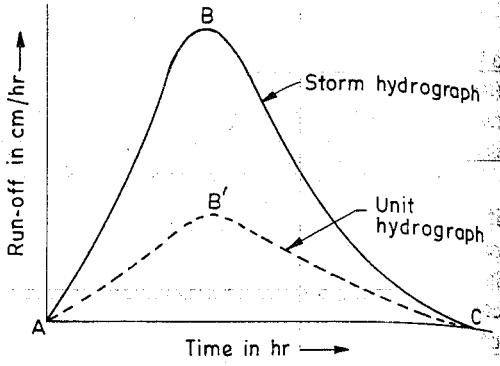


Fig. 7.97

The final step is to assign an effective storm duration from the study of the storm hyetograph. Periods of low rainfall at the beginning and end of the storm are omitted, if they do not contribute substantially to the runoff, as shown in Fig. 7.98.

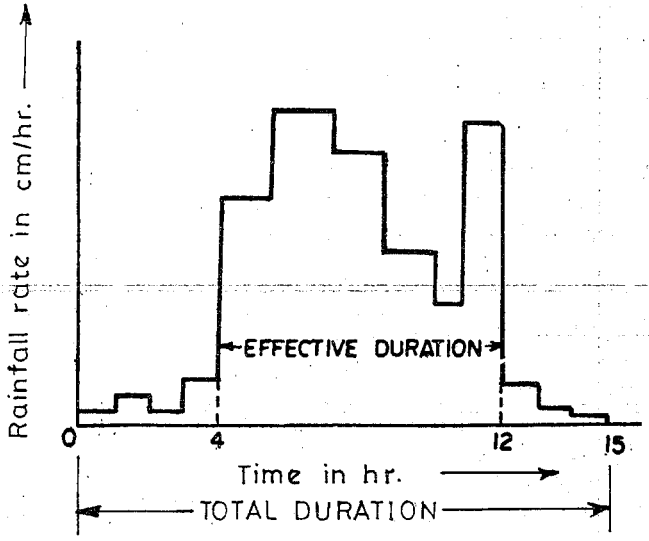


Fig. 7.98. Assigning effective specified durations to rainstorms.

In Fig. 7.98, the duration of rainfall is 15 hours, but still it can be assumed to have an effective duration equal to 8 hours. In this way, the number of durations can be reduced, and the various storms of different durations can be assigned the same unit duration. *Unit storms are, therefore, not the storms of the same durations, but they are the storms of the like durations.*

7.45.3. Preparing Runoff Hydrograph for the Design Storm. The unit hydrograph of a specified unit duration, such as 6 hr U.H. obtained from the past data can be easily used to obtain the hydrograph of future storms of the like durations, provided the volume of runoff resulting from such a storm is estimated. The total volume of run off (in cm) resulting from a storm, when divided by 1 cm, will give us the multiplying factor, *i.e.*

the factor by which the ordinates of the unit hydrograph must be multiplied in order to obtain the ordinates of the direct runoff hydrograph as shown in Fig. 7.99.

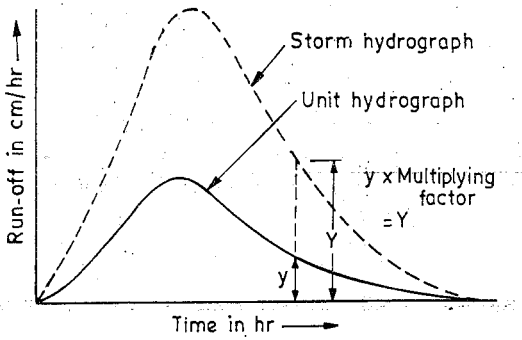


Fig. 7.99

The unit hydrograph of a specified duration (t_1) can also be used to evaluate the runoff hydrographs for storms of *longer durations* (t_2), which are integral multiples of the given duration*. Say for example, a 4 hr unit hydrograph can be used to compute the hydrograph of a storm of 8 hr duration. This is accomplished by dividing the longer storm into two parts. The hydrographs of runoff for each part are computed separately and added. *But care should be taken to see that the hydrograph of the second part of the storm will start 4 hours later than that of the first part.* This will become more clear when we solve a numerical example, a little later.

A tolerance of about 25 per cent of the adopted unit hydrograph duration is generally accepted without serious errors. Thus, a 4 hour-unit hydrograph might be applied to storms of 3 to 5 hours effective durations.

Sometimes, the unit hydrographs having similar range in durations, are averaged to obtain an average unit hydrograph. An average unit hydrograph is preferred to a single storm unit hydrograph, since the averaging tends to minimise errors in the data.

Large variations in the rainfall intensity during the unit period may considerably affect the accuracy of the unit hydrograph approach. Errors due to this reason, can be reduced by using unit hydrographs for relatively short time periods. The short periods of unit hydrographs can be used to develop the hydrograph resulting from a long rain of varying intensity. Experience *has shown that the best unit period is about one-fourth of the basin lag, i.e., the time from the centre of mass of rainfall to the peak of the hydrograph.*

Example 7.32. *In a typical 4 hr. storm producing 50 mm of excess rain from a basin, the following flows in the stream, are recorded.*

Time in hours	*Flow in cumecs*
0	*0.0*
2	*1.22*
4	*4.05*
6	*6.75*
8	*5.67*
12	*3.375*
16	*1.35*
20	*0.0*

(a) Plot the unit hydrograph of run off for this storm.

(b) Estimate, as accurately as possible, the peak flow and the time of its occurrence, in a flood created by a 8 hours storm, which produces 2.5 cm of effective rainfall during

* For determining unit hydrographs of shorter or any other fractional duration, S-curve hydrograph is used, as discussed a little later.

the first 4 hours and 3.75 cm of effective rainfall during the second 4 hours. Assume the base flow to be negligible.

Solution. First of all, the hydrograph of the runoff obtained from the given storm of 4 hour duration is plotted, by using the various given values of time and flow. This is shown by the upper curve (*i.e.* curve 1) in Fig. 7.100.

Since this hydrograph contain 5 cm of runoff volume, we must multiply its ordinates by 1/5, *i.e.* we must divide its ordinates by 5 in order to obtain unit hydrograph of 4 hour-duration. This is shown by the lower curve (*i.e.* curve 2) in Fig. 7.100.

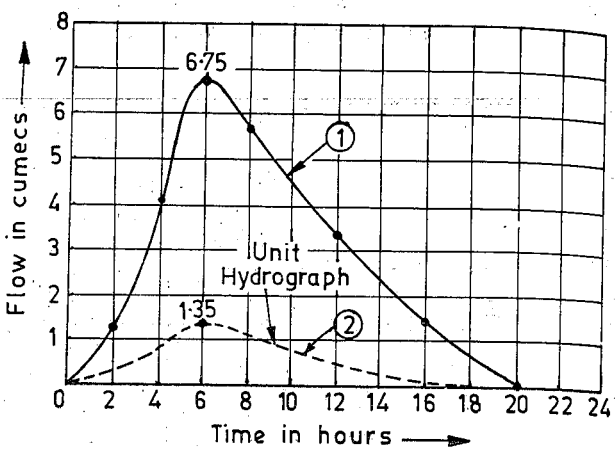


Fig. 7.100

The storm of 8 hours duration has been divided into two parts, each of 4 hours duration. During the first part, 2.5 cm of excess rain is produced; and during the second part, 3.75 cm of excess rain is produced. The S.R. hydrograph for the first part is obtained by multiplying the ordinates of unit hydrograph by 2.5, and that for the second part by multiplying by 3.75.

The first part hydrograph is plotted as curve 1 in Fig. 7.101, and second part as curve 2, but at a lag of 4 hours, as shown. These two hydrographs are added together, so as to obtain the final total hydrograph of 8 hours duration rain. This is shown by curve (3) in Fig. 7.101.

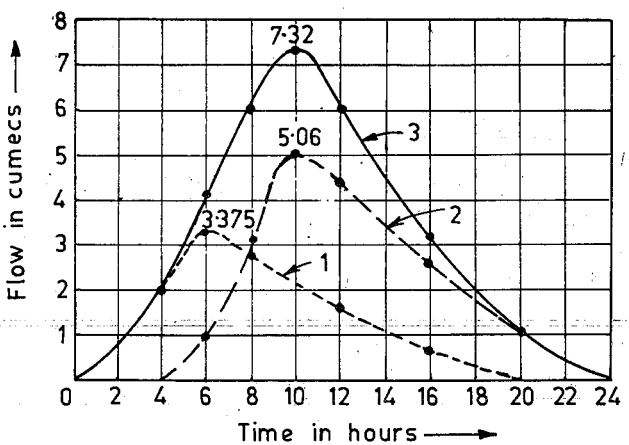


Fig. 7.101

The peak value of total hydrograph is found to be 7.32 cumecs, and it occurs at a time $t = 10$ hours, *i.e.* after 10 hours from the start of rain. **Ans.**

Alternatively, the problem can be solved easily by using an analytical solution, as given in Table 7.40, which is self explanatory.

Table 7.40

Time in hr	*Ordinates of storm hydrograph of 5 cm*	*Ordinates of unit hydrograph of 1 cm = col (2) + 5*	*Ordinates of first 4 hr hydrograph producing 2.5 cm rain = 2.5 × col (3)*	*Ordinates of 2nd 4 hr hydrograph producing 3.75 cm rain at a lag of 4 hr = 3.75 × col (3) (written 2 lines down)*	*Total 8 hr hydrograph ordinates in cumecs = (4) + (5)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0.00	0.00	0.00	—	0.00
2	1.22	0.24	0.61	—	0.61
4	4.05	0.81	2.02	0.00	2.02
6	6.75	1.35	3.37	0.90	4.27
8	5.67	1.15	2.83	3.03	5.86
10	—	—	2.26*	5.06	7.32**
12	3.375	0.68	1.69	4.31	6.00
16	1.35	0.27	0.67	2.55	3.32
20	0.00	0.00	0.00	1.01	1.01
24	—	—	—	0.00	0.00

Example 7.33. *The ordinates of a 3 hour unit hydrograph are given below :*

Time in hr	*0*	*3*	*6*	*9*	*12*	*15*	*18*	*21*	*24*	*27*	*30*
Ordinates m^3/sec	*0*	*10*	*25*	*20*	*16*	*12*	*9*	*7*	*5*	*3*	*0*

Find the ordinates of a 6 hour unit hydrograph for the same basin, analytically. Also sketch this unit hydrograph. What is the peak value of discharge in this unit hydrograph ?

Solution. When the given 3 hr unit hydrograph is added to the same 3 hr unit hydrograph, placed at a lag of 3 hr, then we obtain the ordinates of a 6 hr hydrograph containing 2 cm runoff. Hence, when this graph is divided by 2, we can obtain the

*Interpolated between upper and lower values corresponding to 8 and 12 hours, i.e. $\frac{2.83 + 1.69}{2} = 2.26$

**Peak value = 7.32 cumecs at $t = 10$ hr.

hydrograph of 6 hr duration, containing 1 cm runoff, which will be nothing but 6 hr unit hydrograph.

This addition of 2 unit hydrographs is carried out, in Table 7.41. The ordinates, so obtained, in column (4) are divided by 2, to obtain unit hydrograph ordinates in column (5) of this table, which is self explanatory.

Table 7.41

Time from start in hr	*3 hr unit hydrograph ordinates in m^3/sec*	*3 hr unit hydrograph ordinates lagged by 3 hr in m^3/sec*	*(2) + (3); i.e. ordinates of 6 hr surface runoff hydrograph containing 2 cm of runoff in m^3/sec*	*Ordinates of 6 hr unit hydrograph = $\frac{col.\ (4)}{2}$ in m^3/sec*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
0	0	—	0	0
3	10	0	10	5
6	25	10	35	17.5
9	20	25	45	**22.5**
12	16	20	36	18
15	12	16	28	14
18	9	12	21	10.5
21	7	9	16	8
24	5	7	12	6
27	3	5	8	4
30	0	3	3	1.5
33	—	0	0	0

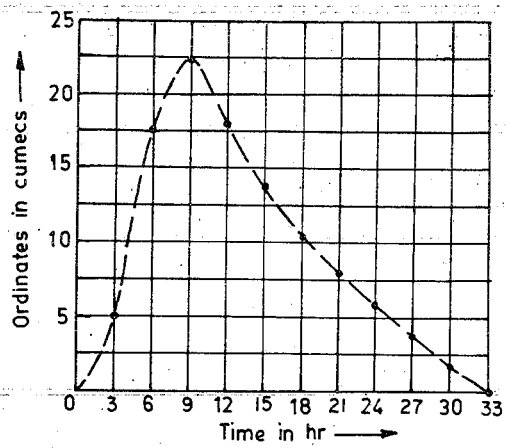


Fig. 7.102

These ordinates are plotted in Fig. 7.102 to obtain the requisite unit hydrograph. The peak value of discharge comes to 22.5 cumecs. **Ans.**

Example 7.34. *Ordinates of 8 hour unit hydrograph for a drainage basin are given in the table below :*

Times in hours	*Ordinates of 8 hr unit hydrograph*
0	*0.0*
4	*5.5*
8	*13.5*
12	*26.5*
16	*45.0*
20	*82.0*
24	*162.0*
28	*240.0*
32	*231.0*
36	*165.0*
40	*112.0*
44	*79.0*
48	*57.0*
52	*42.0*
56	*31.0*
60	*22.0*
64	*14.0*
68	*9.5*
72	*6.6*
76	*4.0*
80	*2.0*
84	*1.0*
88	*0*

Obtain a 24 hour unit hydrograph by tabulation method. Neatly sketch it.

(Engg. Services, 1988)

Solution. If the unit hydrograph of 8 hr duration is added to another 8 hr unit hydrograph placed at a lag of 8 hrs, then surely we will get a surface runoff hydrograph of 16 hr duration containing 2 cm of effective rainfall. Similarly, when 3 unit hydrographs each of 8 hr unit duration, are added together, placed at 8 hr lag successively from one another, then we will get the ordinates of a 24 hr S.R. hydrograph, containing 3 cm runoff. These ordinates, when divided by 3, will give us the ordinates of a unit hydrograph of 24 hr duration.

This method is used to compute the 24 hr unit hydrograph, as shown in Table 7.42. The ordinates of 24 hr unit hydrograph are computed in column (6). Finally, the unit hydrograph is plotted in Fig. 7.103.

Table 7.42

Time in hrs	*1st 8 hr unit hydrograph*	*2nd 8 hr unit hydrograph*	*3rd 8 hr unit hydrograph*	*Total 24 hr hydrograph of 3 cm runoff (2) + (3) + (4)*	*Ordinates of 24 hr unit hydrograph* $= \frac{Col\ (5)}{3}$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0.0			0	0
4	5.5			5.5	1.8
8	13.5	0		13.5	4.5
12	26.5	5.5		32.0	10.7
16	45.0	13.5	0	58.5	19.5
20	82.0	26.5	5.5	114.0	38.0
24	162.0	45.0	13.5	220.5	73.5
28	**240.0**	82.0	26.5	348.0	116.0
32	231.0	162.0	45.0	438.0	146.0
36	165.0	**240.0**	82.0	487.0	162.0
40	112.0	231.0	162.0	**505.0**	**168.3**
44	79.0	165.0	**240.0**	484.0	161.3
48	57.0	112.0	231.0	400.0	133.3
52	42.0	79.0	165.0	286.0	95.3
56	31.0	57.0	112.0	200.0	66.7
60	22.0	42.0	79.0	143.0	47.7
64	14.0	31.0	57.0	102.0	34.0
68	9.5	22.0	42.0	73.5	24.5
72	6.6	14.0	31.0	51.6	17.2
76	4.0	9.5	22.0	35.5	11.8
80	2.0	6.6	14.0	22.6	7.5
84	1.0	4.0	9.5	14.5	4.8
88	0.0	2.0	6.6	8.6	2.9
92	—	1.0	4.0	5.0	1.7
96	—	0.0	2.0	2.0	0.7
100	—	—	1.0	1.0	0.3
104	—	—	0.0	0.0	0.0

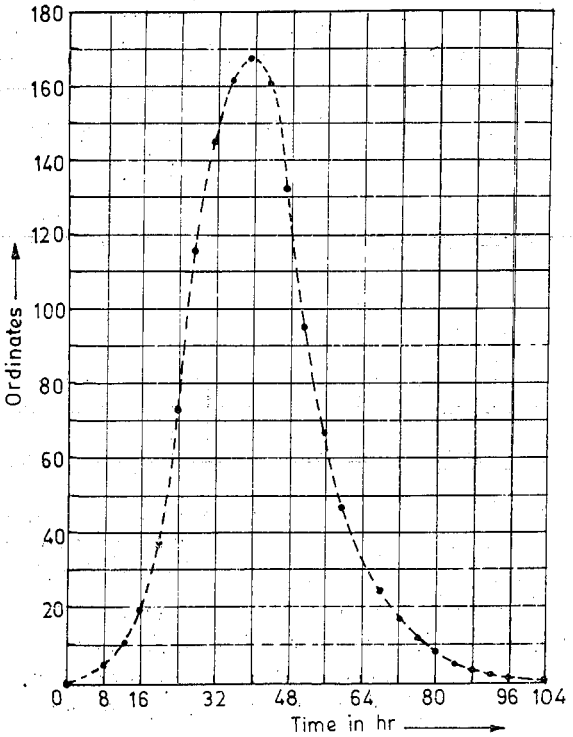


Fig. 7.103. Requisite 24 hr. hydrograph for example 7.34.

Example 7.35. *Using the 3 hr. unit hydrograph given below, find the peak flow, resulting from four successive 3 hour periods of rainfall producing 0.35, 0.87, 1.39 and 0.77 cm of runoff respectively from a basin.*

Time in hour	*0*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*	*14*	*15*
Flow in m^3/s	*0*	*16*	*58*	*173*	*337*	*440*	*400*	*285*	*215*	*165*	*122*	*90*	*60*	*35*	*16*	*0*

Neglect base flow. *(Bhopal University, 1980)*

Solution. The ordinates of hydrographs that would be produced by 0.35, 0.87, 1.39 and 0.77 cm of runoff can be obtained by multiplying the given ordinates of unit hydrograph by these factors, respectively. Now, since all these 4 hydrographs are produced by successive rains, each of 3 hr. unit duration, the total resultant hydrograph can be obtained by summation of the hydrographs of all the four rains, each plotted after 3 hr., successively. This computation can be done mathematically, as shown in Table 7.43, which is self explanatory. The peak rate of runoff = 874.81 cumecs. **Ans.**

Table 7.43. Solution Table for Example 7.35.

Time in hr	*Ordinates of unit hydrograph in cumecs*	*Ordinates of first hydrograph in cumecs; 0.35 col (1)*	*Ordinates of 2nd hydrgoraph in cumecs; 0.87 × col (1) tabulated after 3 hr*	*Ordinates of 3rd hydrograph in cumecs; 1.39 × col (1) tabulated after 6 hr*	*Ordinates of 4th hydrograph in cumecs; 0.77 × col (1) tabulated after 9 hr*	*Ordinates of Total S.R. hydrograph; (3) + (4) + (5) + (6) in cumecs*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
0	0	0	—	—	—	0
1	16	0.35 × 16	—	—	—	5.6
2	58	0.35 × 58	—	—	—	20.3
3	173	0.35 × 173	0	—	—	60.6
4	337	0.35 × 337	0.87 × 16	—	—	131.87
5	440	0.35 × 440	0.87 × 58	—	—	204.46
6	400	0.35 × 400	0.87 × 173	0	—	290.91
7	285	0.35 × 285	0.87 × 337	1.39 × 16	—	315.43
8	215	0.35 × 215	0.87 × 440	1.39 × 58	—	463.42
9	165	0.35 × 165	0.87 × 400	1.39 × 173	0	588.47
10	122	0.35 × 122	0.87 × 285	1.39 × 337	0.77 × 16	771.40
11	90	0.35 × 90	0.87 × 215	1.39 × 440	0.77 × 58	**874.81***
12	60	0.35 × 60	0.87 × 165	1.39 × 400	0.77 × 173	853.76
13	35	0.35 × 35	0.87 × 122	1.39 × 285	0.77 × 337	774.03
14	16	0.35 × 16	0.87 × 90	1.39 × 215	0.77 × 440	721.55
15	0	0	0.87 × 60	1.39 × 165	0.77 × 400	589.55
16	—	—	0.87 × 35	1.39 × 122	0.77 × 285	419.48
17	—	—	0.87 × 16	1.39 × 90	0.77 × 215	304.57
18	—	—	0	1.39 × 60	0.77 × 165	210.45
19	—	—	—	1.39 × 35	0.77 × 122	117.95
20	—	—	—	1.39 × 16	0.77 × 90	91.54
21	—	—	—	0	0.77 × 60	46.20
22	—	—	—	—	0.77 × 35	26.95
23	—	—	—	—	0.77 × 16	12.32
24	—	—	—	—	0	0

*Peak rate of discharge = 874.81 cumecs. **Ans.**

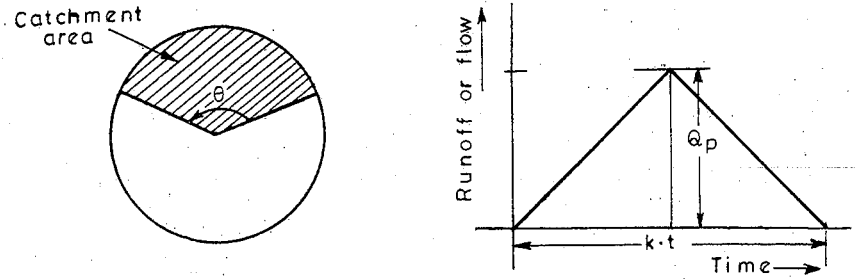
Example 7.36. *A watershed is in the form of a sector of a circle of radius r metres and angle θ radians. A t hr. storm with rainfall excess of x cm, produces a surface runoff hydrograph with linear rising and recession segments, having a base equal to k times the duration. Determine the peak of the surface runoff hydrograph in cumecs.*

(Bhopal University, 1982)

Solution. Area of the sector of the circle of radius r metre

$$= \pi r^2 \cdot \left(\frac{\theta^\circ}{360^\circ}\right)$$

$$= \pi r^2 \cdot \frac{\theta \text{ (radian)}}{2\pi \text{ (radian)}}$$



(*a*) watershed (*b*) hydrograph

Fig. 7.104

$$= \frac{1}{2} \cdot r^2 \cdot \theta \ (m^2) \text{ as } r \text{ is in } m \qquad ...(i)$$

If Q_p is the peak runoff, then

Area of the hydrograph

$$= \frac{1}{2} \cdot k \cdot t \cdot Q_p$$

This area is equal to rainfall excess of x cm

$$\therefore \qquad x = \frac{1}{2} kt\, Q_p$$

$$\therefore \qquad Q_p = \frac{2 \cdot x}{kt} \text{ cm/hr.} \qquad ...(ii)$$

($\because$ x is in cm and t is in hr)

$$\therefore \qquad Q_p = \left(\frac{2x}{kt} \cdot \frac{1}{100}\right) \frac{\text{Area of catchment in m}^2}{60 \times 60} \text{ m}^3/\text{sec}$$

$$= \frac{2x}{kt} \times \frac{1}{100} \times \frac{1}{2} r^2 \cdot \frac{\theta}{60 \times 60} \text{ m}^3/\text{sec}$$

$$= \frac{x \cdot r^2 \cdot \theta}{kt} \frac{1}{3{,}60{,}000} \text{ m}^3/\text{sec.}$$

Hence, the peak of surface runoff hydrograph

$$= \frac{1}{3{,}60{,}000} \left[\frac{x \cdot r^2 \cdot \theta}{kt}\right] \text{cumecs.} \quad \textbf{Ans.}$$

Example 7.37. *The ordinates of a 4-hour unit hydrograph of a catchment are given below :*

Time (h)	*0*	*4*	*8*	*12*	*16*	*20*	*24*	*28*	*32*	*36*	*40*	*44*
Ordinate of 4 h UH (m^3/s)	*0*	*20*	*60*	*150*	*120*	*90*	*70*	*50*	*30*	*20*	*10*	*0*

Derive the flood hydrograph in the catchment due to the storm given below :

Time from start of storm (h)	*0*	*4*	*8*	*12*
Accumulated rainfall (cm)	*0*	*5.0*	*5.8*	*8.8*

The ϕ-index for the catchment can be assumed to be 0.25 cm/h and a constant base flow of 20 m^3/s is appropriate. *(Civil Services, 1991)*

Solution. Let us, first, find excess rain during each interval of 4 hr of the given storm, as computed in table 7.44.

Table 7.44

S.No. of 4 hr Interval	*Time interval in hr from start*	*Cumulative rainfall in cm*	*Rainfall in each interval in cm*	*Infiltration during each interval* in cm*	*Excess rain in each interval in cm*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1	0—4	5.0	5.0	1.0	4.0
2	4—8	5.8	0.8	0.8	Nil
3	8—12	8.8	3.0	1.0	2.0

The calculations for the flood hydrograph are now computed in table 7.45, by working out hydrograph ordinates in the first 4 hr interval by multiplying unit hydrograph ordinates by 4 (*i.e.* excess rain) in col. (3); and those for third interval excess rain, by 2 in col. (4). The ordinates due to 3rd interval excess rain will evidently start from 8 hr instead of normal zero. Additions of these ordinates in col. (5), when further added by base flow of 20 m^3/s, will give us the ordinates of the flood hydrograph, as worked out in col. (6) of Table 7.45. **Ans.**

Table 7.45

Time from start in h	*U.H. ordinates in m^3/s*	*Hydrograph ordinates produced by first 4 hr rain, having excess rain = 5 – 1 = 4 cm; col. (2) × (4) (m^3/s)*	*Hydrograph ordinates produced by 3rd-4 hr rain having excess rain = 3 – 1 = 2 cm col. (2) × 2 with a lag of 8 hr (m^3/s)*	*Total hydrograph col. (3) + col (4) in m^3/s*	*Flood hydrograph ordinates col. (5) + Baseflow of 20 m^3/s in m^3/s*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0	0	—	0	20
4	20	80	—	80	100
8	60	240	0	240	260
12	150	600	40	640	660
16	120	480	120	600	620
20	90	360	300	660	**680**
24	70	280	240	520	540
28	50	200	180	380	400
32	30	120	140	260	280
36	20	80	100	180	200
40	10	40	60	100	120
44	0	0	40	40	60
48	—	—	20	20	40
52	—	—	0	0	20

*0.25 cm/h × 4 h = 1.0 (limited to rainfall value of col. (4).

Example 7.38. *The ordinates of a 3 hr unit hydrograph (U.H.) of a catchment are given below :*

Time (h)	0	3	6	9	12	15	18	21
Ordinates of 3 h U.H. in (m^3/s)	0	10	20	16	12	8	4	0

Derive the flood hydrograph at the catchment outlet due to a storm shown below :

Time from start of storm (h)	0	3	6	9
Accumulated rainfall (cm)	0	3.9	4.7	7.6

Assume ϕ_{index} of the catchment as 0.3 cm/h and a constant base flow of 10 m^3/s.

[AMIE–S. 1993 (New Scheme)]

Solution. Let us first find excess rain during each interval of 3 hr of the given storm, as computed in table 7.46.

Table 7.46

Sr. No. of 3 hr interval	*Time interval in hr from start*	*Cumulative rainfall in cm*	*Rainfall in each interval*	*Infiltration during each interval* in cm*	*Excess rain in each intervals in cm*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1	0—3	3.9	3.9	0.9	3.0
2	3—6	4.7	0.8	0.8	Nil
3	6—9	7.6	2.9	0.9	2.0

The calculations for the flood hydrograph are now computed in table 7.47, as done in the previous example, by using the given unit graph ordinates

Table 7.47

Time from start in h	*U.H. ordinates in m^3/s*	*Hydrograph ordinates produced by first 3 hr rain having excess rain = 3 cm col (2) × 3*	*Hydrograph ordinates produced by the third 3 hr rain having excess rain = 2 cm; col (2) × 2 at a lag of 6 hr*	*Total hydrograph col (3) + col (4) in m^3/s*	*Flood hydrograph ordinates col (5) + base flow of 10 m^3/s in m^3/s*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	0	0	—	0	10
3	10	30	—	30	40
6	20	60	0	60	70
9	16	48	20	68	78
12	12	36	40	76	86
15	8	24	32	56	66
18	4	12	24	36	46
21	0	0	16	16	26
24	—	—	8	8	18
27	—	—	0	0	10

Col. (6) of the above table shows the ordinates of the requisite flood hydrograph w.r. to time values given in col (2). **Ans.**

* 0.30 cm/h × 3h = 0.9 cm [limited to rainfall value of col. (4)]

Example 7.39. *The figures below represent streamflow in cumecs from a 1000 acres drainage area :*

Time in hour	2	4	6	8	10	N 12 Noon	2	4	6
Flow in cumecs	5.1	6.5	7.4	10.2	8.8	7.4	5.9	5.4	4.8

(a) Construct a unit hydrograph for this basin :

(b) Calculate the ϕ_{index} of the storm, if the above streamflow resulted due to a storm precipitation of 4 cm occurring in 4 hours duration.

Solution. A slight deep look at the stream data indicates that the discharge caused by the storm has started arriving over the base flow at about 0 hr (when base flow may be of the order of 4.8 cumecs) and ends at 18 hr (6 PM), when the discharge value again becomes equal to the base flow of about 4.8 cumecs.

Therefore, assuming this base flow to be 4.8 cumecs (as it is only approximate), we tabulate the SRO ordinates, as caused by the storm, w.r.t. to time, in Table 7.48.

Table 7.48

Time (1)	*Total streamflow in cumecs* (2)	*Base flow in cumecs* (3)	*SRO = col (2) – 4.8 in cumecs* (4)
0	4.8 (assumed)	4.8	0
2	5.1	4.8	0.3
4	6.5	4.8	1.7
6	7.4	4.8	2.6
8	10.2	4.8	5.4
10	8.8	4.8	4.0
12	7.4	4.8	2.6
14	5.9	4.8	1.1
16	5.4	4.8	0.6
18	4.8	4.8	0

Using Simpson's rule, the area enclosed by this discharge hydrograph.

$$= \frac{H}{3}\left[\frac{\text{1st} + \text{Last ordinate}}{2} + 4\,\text{Even ordinates} + 2\,\text{Odd ordinates})\right]$$

$$= \frac{2 \times 60 \times 60 \text{ sec}}{3}\left[\frac{0+0}{2} + 4\,(0.3 + 2.6 + 4.0 + 1.1) + 2\,(1.7 + 5.4 + 2.6 + 0.6\right]\frac{\text{m}^3}{\text{sec}}$$

$$= \frac{2 \times 60 \times 60}{3}\,[32 + 20.6]\,\text{m}^3 = \mathbf{52.6 \times 2400\ m^3}$$

Catchment area = 1000 acres

$$= \frac{1000}{2.47} \text{ hectares}$$

$$= \frac{1000}{2.47} \times 10^4\ \text{m}^2 = 404.86 \times 10^4\ \text{m}^2$$

$\therefore$ Depth of water in the hydrograph

$$= \frac{\mathbf{52.6 \times 2400}}{404.86 \times 10^4}\ \text{m} = 3.118\ \text{cm.}$$

Since the given hydrograph encloses 3.118 cm of rain, we can obtain the hydrograph containing 1 cm of rain (*i.e.* unit hydrograph) by dividing the ordinates of this given Direct Runoff Hydrograph by 3.118, as shown in col. (3) of Table 7.49.

Table 7.49

Time, hr	*Ordinates of DRH in cumecs*	*Ordinates of unit hydrograph* $= \frac{\text{Col. (2)}}{3.118}$ *in cumecs*
(1)	*(2)*	*(3)*
0	0	0
2	0.3	0.096
4	1.7	0.545
6	2.6	0.834
8	5.4	1.732
10	4.0	1.283
12	2.6	0.834
14	1.1	0.353
16	0.6	0.192
18	0	0

(*b*) When the above storm runoff was caused by a rainfall of 4 cm in 4 hr, we have

$$\text{rainfall} = 4 \text{ cm}$$

$$\text{runoff (excess rainfall)} = 3.118 \text{ cm}$$

$\therefore$ Infiltration including basin recharge

$$= 4 - 3.118 = 0.882 \text{ cm}$$

$$t_r = 4 \text{ hr (given)}$$

$$\therefore \quad \phi_{index} = \left(\frac{0.882}{4}\right)^{*} = 0.22 \text{ cm/hr.} \quad \textbf{Ans.}$$

Example 7.40. *The rainfall runoff relation for the 662.4 sq. km of a drainage basin under dry soil conditions, is as follows :*

Rainfall in cm	*Runoff in cm*
1.5	*0.0*
5.0	*2.0*
7.5	*4.0*
10.0	*6.5*
12.5	*8.75*

A storm of 2 hours effective duration occurred over this basin at time when the soil was dry. The resulting hydrograph was as follows :

* Strictly speaking, it is W_{index} but is approximately taken as ϕ_{index} here.

Time, hr	*Flow, cumecs*
0	*0*
3	*300*
6	*500*
9	*400*
12	*250*
15	*75*
18	*0*

Determine the number of mm of runoff from the basin resulting from this storm. Assume zero base flow. Approximately how much rain was there in this storm. What was the average coefficient of runoff during this storm? Predict the peak flow that would result from a 4 hour storm during which there were 12.5 cm of excess rainfall. Once again assume dry soil conditions, i.e. zero base flow. Plot the resulting hydrograph showing the approximate time of peak flow. Simpson's rule may be used for evaluating approximately the area of the hydrograph.

Solution. First of all, the relation between runoff and rainfall is plotted by using the various observed values of rainfall and runoff. The curve obtained is shown in Fig. 7.105.

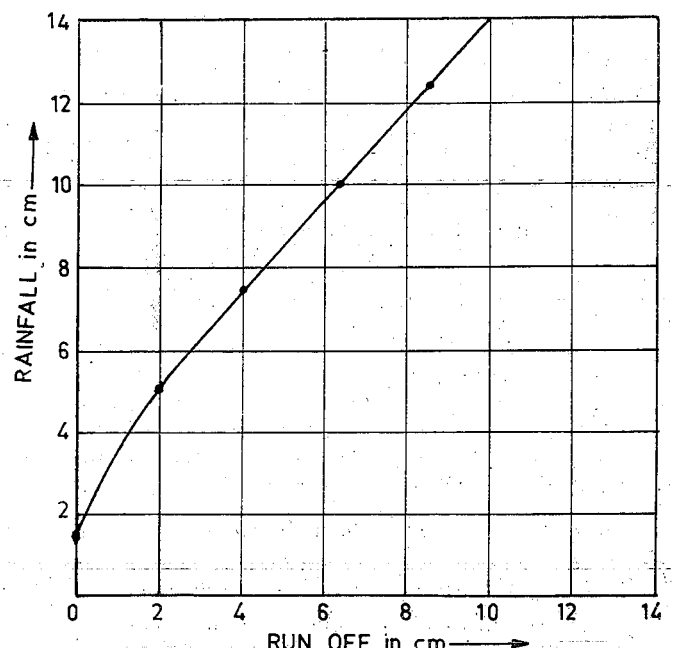


Fig. 7.105

Then, the hydrograph of the given storm of 2 hour effective duration is plotted. This is shown by curve (1) of Fig. 7.106.

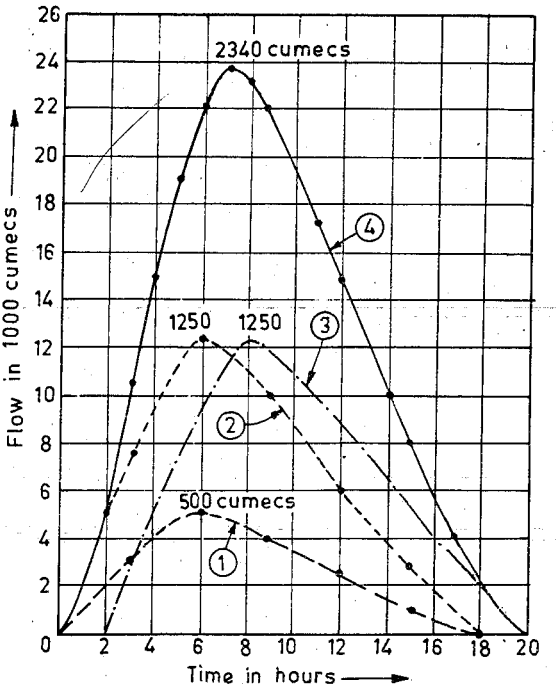


Fig. 7.106

The area under this graph can be calculated approximately by using Simpson's rule : which states

$$A = \frac{H}{3}\,[(\text{First ordinate} + \text{Last ordinate}) + 2\,(\text{Odd ordinates}) + 4\,(\text{Even ordinates})]$$

$$= \frac{H}{3}\left[(y_0 + y_n) + 2\,(y_2 + y_4 + y_6 + \ldots)^* + 4\,(y_1 + y_3 + \ldots)\right] \qquad \ldots(7.113)$$

Various values of the hydrograph ordinates are given at every 3 hours interval. They are 0, 300, 500, 400, 250, 75, 0 cumecs.

$\therefore$ Area under the graph

$$= \frac{3 \text{ hr.}}{3}\left[(0 + 0 + 2\,(500 + 250) + 4\,(300 + 400 + 75)\right]$$

$$= \frac{3 \times 60 \times 60\,(\text{sec})}{3}\,[1500 + 3100]\,\frac{\text{m}^3}{(\text{sec})}$$

$$= 3600\,[4600]\,\text{m}^3 = 16.56 \times 10^6\,\text{m}^3$$

$$= \frac{16.56 \times 10^6}{662.4 \times 10^6\,(\textit{i.e.}\ \text{area of basin in m}^2)}\ \text{in metres}$$

$$= \frac{16.56 \times 10^6}{662.4 \times 10^6} \times 100\ \text{cm} = 2.5\ \text{cm} \quad \textbf{Ans.}$$

It means that this storm hydrograph is enclosing 1 inch = 2.5 cm of runoff, and hence; it can be considered as a unit hydrograph of 1 inch. *i.e.* 2.5 cm of rain excess. From Fig. 7.105, the rainfall corresponding to a runoff equal to 2.5 cm can be read out as 5.8 cm. **Ans.**

---

* y_2 is the third ordinate, y_4 is the fifth ordinate ; thus y_2, y_4, y_6 ... are the odd ordinates.

Average co-efficient of runoff during this storm

$$= K = \frac{Q}{P} = \frac{2.5}{5.8} = 0.431 \quad \textbf{Ans.}$$

The storm of 4-hour duration can be divided into two parts each of 2-hour duration and each having equal runoff volume

$$= \frac{12.5 \text{ cm}}{2} = 6.25 \text{ cm}.$$

To obtain the ordinates of the hydrograph of each part, the ordinates of the unit hydrograph [curve 1. Fig. 7.106] must be multiplied by $\frac{6.25}{2.5} = 2.5$. But the second part hydrograph must be plotted at a lag of 2 hours. These two hydrographs are shown by curve (2) and curve (3) of Fig. 7.106, respectively. They are added together, so as to obtain the total hydrograph resulting from the given storm. This is shown by curve (4) of Fig. 7.106. From this curve, the *peak value* can be read as **2,340 cumecs,** and that it occurs after **7 hours** from the start. **Ans.**

Example 7.41. *The ordinates of a flood hydrograph resulting from two successive storms each of 1 cm rainfall excess and 6 hour duration are tabulated below. Find a 6 hr. unit hydrograph.*

Time (hour)	*Ordinates of 6 hr flood hydrograph (m^3/s)*
0	10
6	30
12	90
18	220
24	280
30	220
36	166
42	126
48	92
54	62
60	40
66	20
72	10

Solution. This question is the reverse type of solved example 7.38, and can be solved by assuming ordinates of 6 hr. unit graph as $a, b, c, d, e, f, \ldots$ etc. and then computing the flood graph caused by two successive effective rainfalls amounting to 1 cm and 1 cm, respectively. The second rainfall occurs at 6 hr lag. The additive hydrograph ordinates will equal the flood graph ordinates minus base flow, which apparently is 10 m^3/s. The computations are given in table 7.50.

Table 7.50. Solution Table for Example 7.41

(1)	*Total flood graph ordinates (given) minus base flow of 10 m^3/s* (2)	*Assumed ordinates of U.H. m^3/s* (3)	*Flood graph ordinates caused by 1st effective rainfall of 1 cm = 1 × col. (3)* (4)	*Flood graph ordinates caused by 2nd effective rainfall of 1 cm = 1 × col. (3)* (5)	*Total flood graph = col. (4) + (5) = (2)* (6)
0	10 – 10 = 0	0	0		0 = 0
6	30 – 10 = 20	a	a	0	$a = 20$
12	90 – 10 = 80	b	b	a	$b + a = 80$; $\therefore b = 60$
18	220 – 10 = 210	c	c	b	$c + b = 210$; $c = 150$
24	280 – 10 = 270	d	d	c	$d + c = 270$; $d = 120$
30	220 – 10 = 210	e	e	d	$e + d = 210$; $e = 90$
36	166 – 10 = 156	f	f	e	$f + e = 156$; $f = 66$
42	126 – 10 = 116	g	g	f	$g + f = 116$; $g = 50$
48	92 – 10 = 82	h	h	g	$h + g = 82$; $h = 32$
54	62 – 10 = 52	i	i	h	$i + h = 52$; $i = 20$
60	40 – 10 = 30	j	j	i	$j + i = 30$; $j = 10$
66	20 – 10 = 10	k	k	j	$k + j = 10$; $k = 0$
72	10 – 10 = 0	l			

The ordinates of U.H. of 6 hr duration are thus determined in col. (6) as 0, 20, 60, 150, 120, 90, 66, 50, 32, 20, 10, 0 m^3/s at time 0, 6, 12, 18, 24, 30, 36, 42, 48, 54, 60, 66 hr. **Ans.**

7.46. Bernard's Distribution Graph and its Derivation from the Runoff Hydrograph of a Unit Storm

A distribution graph is somewhat similar to a unit hydrograph in the sense that it is a plot having the same time scale on x-axis as in a unit hydrograph, but the ordinates on y-axis are not the S.R.O. rates as in a unit hydrograph, but is the percentage of total S.R.O. that occurred during successive arbitrarily chosen time intervals. This graph is more rational than a unit hydrograph in the sense that *all the unit storms irrespective of their magnitude, produce almost identical distribution graphs.* Now, once a D-graph is derived for a drainage basin (from the runoff hydrograph of a unit storm), it serves as a means of converting any expected volume of surface runoff (*i.e.* excess or effective rainfall) into hydrograph of river discharge.

It can also be stated here that the time interval chosen for the D-graph percentages, has no bearing in relation to the duration of rainfall excess, and the magnitude of this interval depends entirely upon how accurate a reproduction of the hydrograph is desired, and upon the time available for computations. Usually, this interval is taken as 0.1 times the total period of the hydrograph (*i.e.* the base).

For obtaining the distribution graph, therefore, the hydrograph of unit storm must be divided into a suitable number of time intervals.

The area under each strip is calculated, which is expressed as percentage of the total area under the unit graph.

Say for example, in Fig. 7.107, the unit hydrograph is divided into 10 intervals each of 10 minutes unit duration, and let areas under these strips be a_1, a_2, a_3 ..., a_{10} and let the total area be $a_1 + a_2 + \ldots a_{10} = A$.

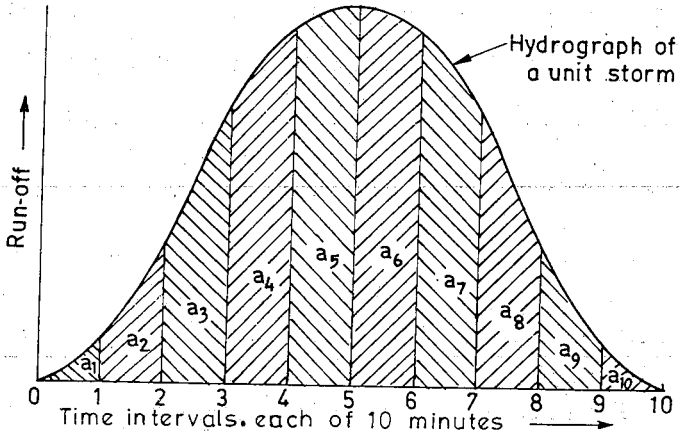


Fig. 7.107

Then, y_1 = 1st ordinate of distribution graph $= \dfrac{a_1 \times 100}{(A)}$

y_2 = 2nd ordinate of distribution graph $= \dfrac{a_2 \times 100}{(A)}$

y_3 = 3rd ordinate of distribution graph $= \dfrac{a_3 \times 100}{(A)}$

.....

y_{10} = 10th ordinate of distribution graph $= \dfrac{a_{10} \times 100}{(A)}$

The ordinates $y_1, y_2, y_3, \ldots, y_{10}$ are plotted on y-axis against the corresponding strip (centre point of each strip). They are joined together, so as to obtain the distribution graph.

The simplest method to calculate $y_1, y_2, y_3, \ldots y_{10}$ in the above case, would be, to measure the ordinate at the centre point of each strip and to divide each by their sum. Let q_{0-1} be the ordinate of S.R.O. between 0 and 1, and similarly, q_{1-2} be the ordinate of S.R.O. between 1 and 2, and so on, then

$$y_1 = \frac{q_{0-1} \times 100}{q_{0-1} + q_{1-2} + \ldots + q_{9-10}}$$

$$y_2 = \frac{q_{1-2} \times 100}{q_{0-1} + q_{1-2} + \ldots + q_{9-10}}$$

$$y_3 = \frac{q_{2-3} \times 100}{q_{0-1} + q_{1-2} + \ldots + q_{9-10}}$$

.....

$$y_{10} = \frac{q_{9-10} \times 100}{q_{0-1} + q_{1-2} + \ldots + q_{9-10}}$$

Since q_{0-1}, q_{1-2}, q_{2-3}, etc. will, in the above case, be the S.R.O. in cumecs corresponding to time intervals of 5, 15, 20 ... minutes; and hence even plotting of unit hydrograph is not required, if the S.R.O. ordinates are given corresponding to the mid-intervals, as explained in the following examples.

Example 7.42. *Determine the distribution graph and its peak percentage for a drainage basin 'A' from a rain that fell on it on 8th May, 1969 for 28 minutes. The discharge observations, consequent to this rain, have been made at every 10 minutes intervals, as tabulated in the Table given below. The period of rise was found to be 30 minutes.*

Time from start in minutes	*S.R.O. in cumecs*
5	0.02
15	0.27
25	0.84
30	0.94
35	0.84
45	0.57
55	0.35
65	0.22
75	0.16
85	0.10
95	0.06
105	0.04
115	0.02

The area of the basin was 30 hectares (0.3 sq kilometres). Determine also the surface runoff in cm depth of water on the basin, that resulted from this rain.

Solution. By looking at the discharge observations, it can be easily concluded that the discharge becomes maximum at a time $t = 30$ minutes. Therefore, the period of rise is 30 minutes. But the period of the storm is 28 minutes, which is less than the period of rise of 30 minutes. Therefore, it can be easily regarded as a unit storm, and hence, the distribution graph can be derived.

The given SRO ordinates (Q) at every 10 minutes interval are tabulated in col. (2) of Table 7.50; their summation is 4.43. The ordinates of the distribution graph or the distribution graph percentages can then be obtained by dividing each ordinate by their sum and multiplying each by 100, as computed in col. (3) of Table 7.51.

Table 7.51

Time from start in minutes (1)	*S.R.O. ordinates or Q in cumecs* (2)	*Ordinates of Distribution graph, i.e.* $y = Q \times 100/\Sigma Q$, *i.e. Distribution graph percentages* (3)
5	0.02	$y_1 = 0.02 \times 100/4.43 = 0.48\%$
15	0.27	$y_2 = 0.27 \times 100/4.43 = 6.08\%$
25	0.84	$y_3 = 0.84 \times 100/4.43 = 18.95\%$
30	0.94	$y_4 = 0.94 \times 100/4.43 = \mathbf{21.23\%}$*
35	0.84	$y_5 = 0.84 \times 100/4.43 = 18.95\%$
45	0.57	$y_6 = 0.57 \times 100/4.43 = 12.86\%$
55	0.35	$y_7 = 0.35 \times 100/4.43 = 7.81\%$
65	0.22	$y_8 = 0.22 \times 100/4.43 = 4.95\%$
75	0.16	$y_9 = 0.16 \times 100/4.43 = 3.67\%$
85	0.10	$y_{10} = 0.10 \times 100/4.43 = 2.23\%$
95	0.06	$y_{11} = 0.06 \times 100/4.43 = 1.36\%$
105	0.04	$y_{12} = 0.04 \times 100/4.43 = 0.95\%$
115	0.02	$y_{13} = 0.02 \times 100/4.43 = 0.48\%$
	$\Sigma Q = 4.43$	$\Sigma y = 100.00\%$

The sum of these ordinates (Σy) should be 100. If any small difference is found, then, it should be adjusted with common sense. The distribution graph, can then be plotted, as shown in Fig. 7.108.

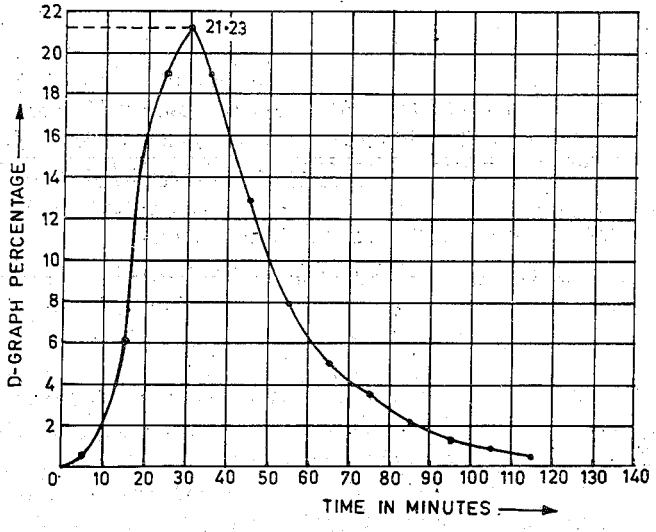


Fig. 7.108

Now, Depth of Surface Runoff

$$= \frac{\Sigma Q \times \Delta t \text{ in sec}}{\text{Area of basin}} = \left[\frac{4.43 \times (10 \times 60)}{30 \times 10^4}\right] \text{m} = \frac{0.886}{100} \times 100 \text{ cm} = 0.886 \text{ cm}.$$

Alternative units for the ordinates of the distribution graph. Sometimes, alternative units 'cubic metre per second per square kilometre per metre of surface runoff' are used and found convenient.

* The distribution graph peak is 21.23 per cent, based on 10 minute time interval.

Example 7.43. *Solve the previous example by determining the ordinates of the Distribution graph in cumecs/sq. kilometre/metre of S.R.O.*

Solution. Total surface runoff expressed in metre depth of water on the basin $= D = 0.886$ cm $= 0.00886$ m (as worked out in the previous example) and area of the basin is 30 hectares, *i.e.*, 0.3 sq. kilometre.

The various ordinates of distribution graph can be obtained by dividing Q (in cumecs) by the depth of water (in metres) and by area A (in sq. kilometres). These ordinates are worked out in column (3) of Table 7.52.

Table 7.52

Time from start in minutes	*Discharge Q in cumecs*	$y = \text{Ordinates of } D \text{ graph} = \frac{Q}{D \times A} = \frac{Q}{0.00886 \times 0.3} = \frac{Q}{0.002658} = 378\,Q$
(1)	(2)	(3)
5	0.02	8
15	0.27	102
25	0.84	316
30	0.94	**354** (Peak value)
35	0.84	316
45	0.57	214
55	0.35	132
65	0.22	83
75	0.16	62
85	0.10	38
95	0.06	23
105	0.04	15
115	0.02	8

The various ordinates of distribution graph, as obtained above, are plotted in Fig. 7.109, so as to obtain the required distribution graph. The maximum ordinate is 354

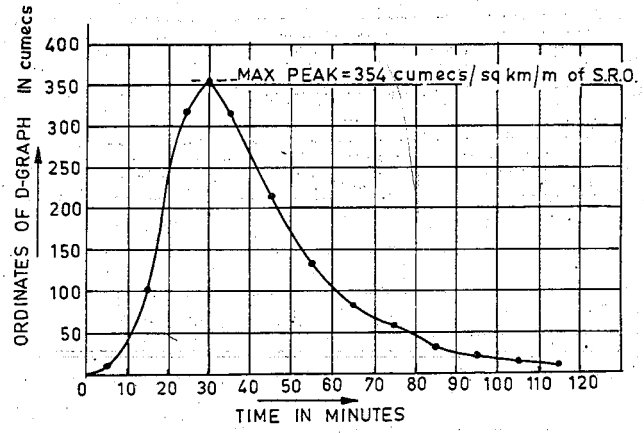


Fig. 7.109

cumecs/sq km/m of S.R.O., based on 10 minutes time interval. **Ans.**

Example 7.44. *Various values of S.R.O. at uniform time intervals that resulted from a unit storm are tabulated below. Determine the distribution graph and the value of its peak ordinate in cumecs/sq km/m depth of S.R.O. The area of the basin is 25 hectares.*

S. No. of 10 minutes interval	*Av. rate of S.R.O. in cumecs*
1	*0.0065*
2	*0.0915*
3	*0.2570*
Mid-point between 3 and 4	*0.4060*
4	*0.3830*
5	*0.2614*
6	*0.1250*
7	*0.0697*
8	*0.0393*
9	*0.0218*
10	*0.0130*
11	*0.0065*
12	*0.0023*

Solution. $D = \text{Depth of S.R.O.} = \dfrac{\text{Total water flow}}{\text{Area of basin}}$

$$D \text{ in m} = \left[\frac{\text{Summation of all discharges (in cumecs) during interval of 10 minutes each}}{\text{Area of basin in sq. m}}\right] \times 10 \times 60 \text{ sec}$$

$$= \frac{1.6830 \times 10 \times 60}{25 \times 10^4} \text{ m} = 0.00405 \text{ m}.$$

The ordinates of distribution graph in cumecs/sq km/m of S.R.O. can be obtained by dividing average rate of S.R.O. by area in sq. km and depth D in m.

Therefore, the values of y are tabulated in column (3) of Table 7.53 by dividing the values of Q by an amount equal to 0.25 (*i.e.* area in sq. km) $\times 0.00405 = \dfrac{1}{987}$: or $y = 987\,Q$.

Table 7.53

Time interval (1)	*Av. rate of S.R.O. in cumecs* (2)	*Ordinates of D-graph* $y = 987\,Q$ (3)
1	0.0065	6.41
2	0.0915	90.4
3	0.2570	253.6
Mid-point between 3 and 4	0.4060	**401.0 (Peak value)**
4	0.3830	377.8
5	0.2614	258.0
6	0.1250	123.2
7	0.0697	68.8
8	0.0393	38.8
9	0.0218	21.5
10	0.0130	12.8
11	0.0065	6.41
12	0.0023	2.27

These ordinates are plotted in Fig. 7.110, so as to obtain the required distribution graph. The peak ordinate is 401 cumecs/sq km/m of S.R.O., based on 10 minutes time interval.

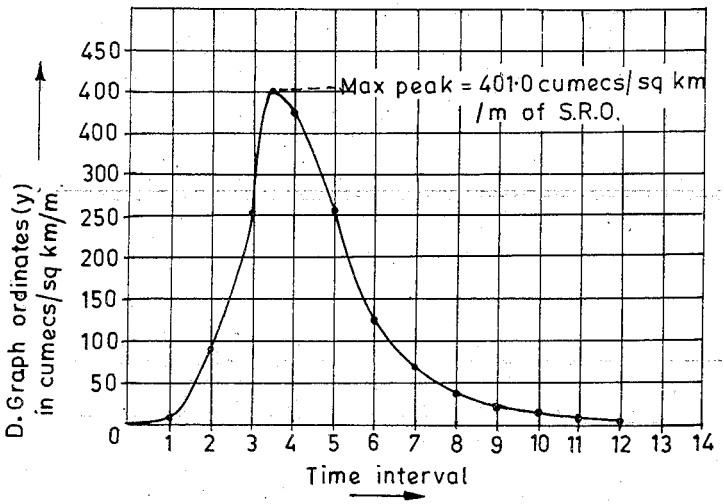


Fig. 7.110

7.46.1. Separation of the Base Flow from Discharge Hydrograph. In fact, *before deriving a distribution graph or a unit hydrograph, it is necessary to separate the base flow from the flow hydrograph of a stream.* In other words, the surface runoff and ground water discharge must, first of all, be separated.

The method of separating the ground water discharge from the runoff has been discussed earlier, and the line representing the base flow must be drawn in that manner, on the runoff hydrograph curve. Generally, the point of beginning of S.R.O. can be easily detected, because of the abrupt fashion in which the hydrograph rises at that point. While, the end point can be detected from the fact that at this stage, there is a sudden break in the slope of the recessional curve.

Sometimes, when other methods for detecting the starting point and end point fail, these points may be arbitrarily chosen, depending upon the personal judgement.

When two overlapping unit hydrographs are involved, they are also separated into single hydrographs. This is illustrated in Fig. 7.111. The recession side of the 1st hydrograph is assumed to be the same as that of the 2nd hydrograph.

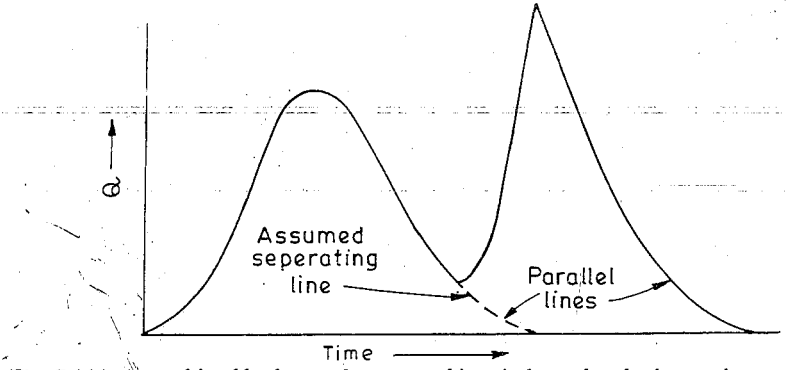


Fig. 7.111. A combined hydrograph separated into independent hydrographs.

7.46.2. Distribution Graphs for Different Water-sheds. The distribution graphs for different unit storms and for a given basin have been found to exhibit a marked similarity, and are almost identical. Thus a typical distribution graph for a given drainage basin is derived and kept in the office, so as to re-use it, in order to predict the runoff in future.

In actual practice, distribution graph derived above is used not only to predict the runoff from the drainage basin for which it is derived, but also for any other drainage basins having almost similar physical characteristics.

Various experiments have been performed, and it has been found that the area of the basin exerts a considerable influence on the shape of the unit hydrograph, although it is not the only factor. Other factors like slope, shape, stream density, etc. do affect it.

Table 7.54

Area of the basin in hectares (1)	*Period of rise in minutes* (2)	*Length of base in minutes* (3)	*Peak percentage based on intervals equal to 0.1 of base* (4)	*Duration of chosen time interval for D-graph in minutes* (5)
29	27	100	32.0	10
30	26	120	38.1	12
49	27	130	34.2	13
121	31	200	37.7	20
149	42	210	37.6	21
368	73	300	31.7	30
1,028	63	290	34.1	29
1,830	104	560	38.9	56
2,140	110	580	37.9	58

The distribution graphs drawn for various basins of different areas show a marked similarity, when the base of the unit hydrograph is divided into equal number of intervals. Their peak values are found to be almost equal, as shown in Table 7.54.

But the peak percentages based on uniform time intervals, say (10 minutes interval) are not uniform, and found to vary inversely with the area. The values were reducing as the area of the basin was increasing, as shown in Table 7.55.

Table 7.55

Basin area in hectares	29	30	49	121	149	368	1,028	1,830	2,140
Peak percentage based on 10 min. interval	32.6	30.4	25.1	18.3	17.2	10.4	8.1	6.8	6.1

Graphs are plotted between the peak percentage (based on uniform time intervals, say 10 min) and the area of the basin, as shown in Fig. 7.112.

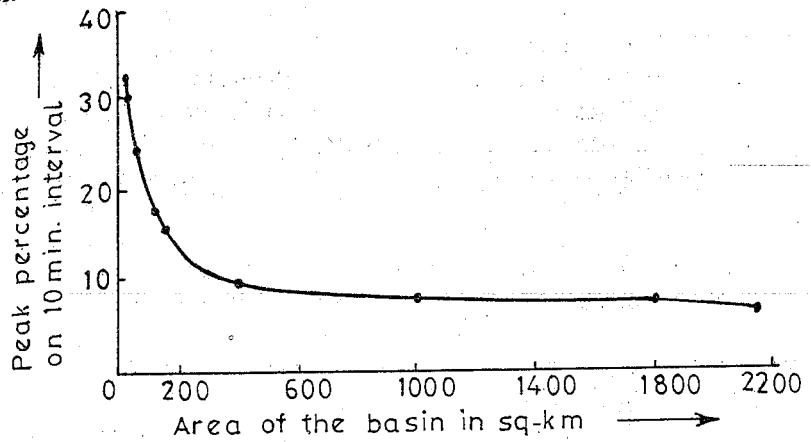


Fig. 7.112

7.46.3. Computation of Runoff Hydrograph from Excess Rainfall By Using D. Graph. The distribution graph can be easily used to compute the hydrograph of direct runoff or peak runoff rate, if the effective rainfall (excess rainfall) is known. This can be done by a single application of the D. graph, if the duration of the effective rainfall is less than the critical rainfall duration, which is usually considered equal to the period of rise for smaller catchments (upto 2500 hectares or so). This use of D. graph will become more clear when we solve the following numerical examples. However, the hydrograph, resulting from a rain of longer duration (*i.e.* grater than the period of rise) must be derived by successive applications of D. graph to unit durations of effective rainfall.

Example 7.45. *An intense rain assumed to be falling at a uniform rate of 5 cm/hr for a period of 80 minutes, on a drainage basin having an area equal to 550 hectares. The average infiltration capacity during the entire rain period has been worked out to be 1.4 cm/hr. Determine the maximum runoff rate, if the peak percentage based on 10 minutes interval from the distribution graph for this basin is 18%.*

Solution. Rain falls at a constant rate of 5 cm/hr ; out of which, infiltration takes place at a constant rate of 1.4 cm/hr. The remaining water appears as excess rain.

$\therefore$ Excess rainfall in cm depth

$$= (5.0 - 1.4) \times \frac{80}{60} = 3.6 \times \frac{4}{3} = 4.8 \text{ cm}$$

Total excess rainfall volume

$$R_e = R_e \text{ in m} \times \text{Area in m}^2$$

$$= \frac{4.8}{100} \times (550 \times 10^4)\ \text{m}^3 = 4.8 \times 550 \times 10^2\ \text{m}^3.$$

Since the peak percentage of D. graph, is based on 10 minutes interval*, the volume of water (R_e) must, first of all, be converted into m^3/sec (cumecs), by dividing by the number of seconds in 10 minutes interval.

* Unit duration.

Therefore, R_e in cumecs

$$= \frac{4.8 \times 550 \times 10^2}{10 \times 60} \text{ m}^3/\text{sec (for 10 minutes duration)}$$

$$= 440 \text{ m}^3/\text{sec (for 10 minutes duration)}$$

The peak percentage from the distribution graph, based on 10 minutes unit duration = 18% (given)

Therefore, peak rate of runoff

$$= 440 \text{ m}^3/\text{s} \times 18/100 = 79.2 \text{ m}^3/\text{sec}$$

Hence, the maximum runoff rate = 79.2 cumecs. **Ans.**

Example 7.46. *The typical rain intensity graph of 60 minutes rain falling over a drainage basin having an area equal to 600 hectares, is shown in Fig. 7.113. The period*

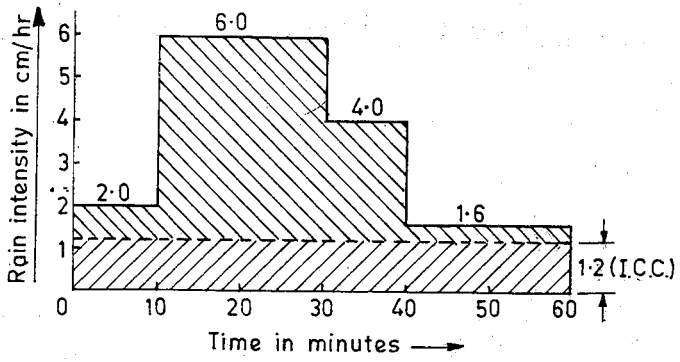


Fig. 7.113

of rise was 50 minutes. The average infiltration capacity is 1.2 cm/hr during the whole rainfall period. Determine the maximum runoff rate, if the peak percentage based on 15 minutes interval from the distribution graph for a similar drainage basin of the same area, was found to be 17.5%.

Solution. The area under the rain hyetograph

$$= 2 \times 10/60 + 6 \times 20/60 + 4 \times 10/60 + 1.6 \times 20/60$$

$$= \frac{1}{6}(2 + 12 + 4 + 3.2) = \frac{1}{6}(21.2) = 3.53 \text{ cm.}$$

The area under I.C.C. $= 1.2 \times 60/60 = 1.2$ cm

The net area $= 3.53 - 1.2 = 2.33$ cm

R_e = Excess rain in cubic metres

$$= \left(\frac{2.33}{100} \text{ m}\right) \times (600 \times 10^4 \text{ m}^2)$$

$$= 13.98 \times 10^4 \text{ m}^3$$

Since the peak percentage of D. graph is based on 15 minutes interval, therefore, the volume of water should, first of all, be converted into cumecs for 15 minutes interval.

R_e in cumecs for 15 minutes.

$$= \frac{13.98 \times 10^4 \text{ m}^3}{15 \times 60 \text{ (sec)}} = 153.3 \text{ m}^3/\text{sec (for 15 minutes duration)}$$

The peak percentage from distribution graph = 17.2% (given)

Therefore, the peak runoff rate = 155.3 m^3/sec × 17.2/100 = 26.7 cumecs. **Ans.**

Example 7.47. *Determine the flood peak resulted from a 10 year rain on a 1200 hectares water-shed 'A'. The intensity duration curves were available in the office, and a graph between the area and period of rise was available, as shown in Fig. 7.114.*

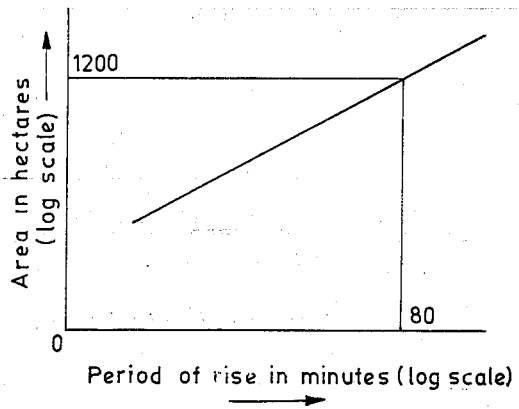


Fig. 7.114. Graph between area and period of rise.

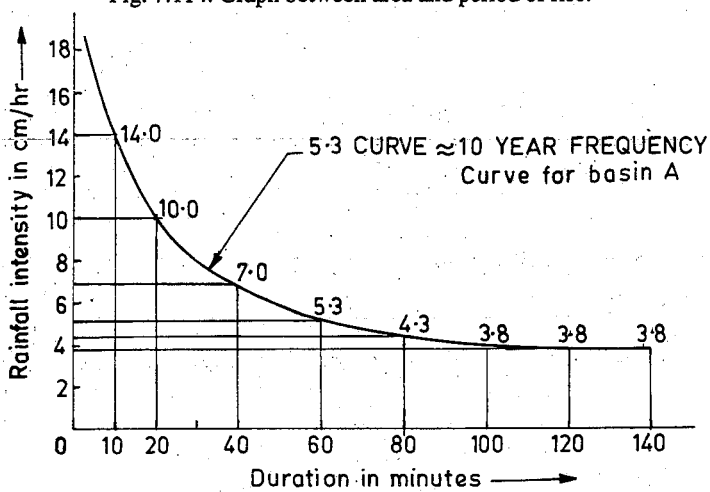


Fig. 7.115. Intensity duration curve.

The given 5.3 curve (Fig. 7.115) can be used to determine the average intensity of a 10 year frequency rain for the basin 'A'. The average infiltration capacity is assumed to be constant and equal to 1.2 cm/hr. The distribution graph for the basin shows the value of its peak percentage based on 10 minutes interval as 8.2 per cent.

Solution. First of all, we should determine the period of rise for basin area equal to 1200 hectares from the graph between the area and period of rise (Fig. 7.114). The value is seen to be 80 minutes. Hence, the critical rainfall duration is 80 minutes.

Now, we should determine the maximum rainfall of 80 minutes duration having a frequency of 10 years. Since 10 years frequency curve is represented by 5.3 curve on the Intensity— Duration curve, we should read the various values of rainfall intensity (average) corresponding to various values of time from Fig. 7.115, as tabulated in Table 7.56.

Table 7.56

Time in minutes	*Rainfall intensity in cm/hr (average) read from graph of Fig. 7.115*
10	14.0
20	10.0
40	7.0
60	5.3
80	4.3

The rainfall excess is now to be calculated, as shown in Table 7.57.

Time from start in minutes	*Average rainfall intensity from graph in cm/hr*	*Accumulated rain in cm i.e. av. rate × time*	*Time increment in minutes*	*Rain during each interval in cm*	*Average rainfall intensity during interval in cm/hr*	*f in cm/hr*	*Excess rainfall rate in cm/hr ; ΔP_e*	*Excess rainfall in each interval ; ΔP_e in cm*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
0	0	0						
10	14.0	$14 \times \frac{10}{60} = 2.33$	10	2.33	$2.33 \times \frac{60}{10} = 13.98$	1.2	12.78	$12.78 \times \frac{10}{60} = 2.13$
20	10.0	$10 \times \frac{20}{60} = 3.33$	10	1.00	$1.0 \times \frac{60}{10} = 6.00$	1.2	4.8	$4.8 \times \frac{10}{60} = 0.80$
40	7.0	$7.0 \times \frac{40}{60} = 4.67$	20	1.34	$1.34 \times \frac{60}{20} = 4.02$	1.2	2.82	$2.82 \times \frac{20}{60} = 0.94$
60	5.3	$5.3 \times \frac{60}{60} = 5.30$	20	0.53	$0.53 \times \frac{60}{20} = 1.59$	1.2	0.39	$0.39 \times \frac{20}{60} = 0.13$
80	4.3	$4.3 \times \frac{80}{60} = 5.73$	20	0.43	$0.43 \times \frac{60}{20} = 1.29$	1.2	0.09	$0.09 \times \frac{20}{60} = 0.03$
								$\Sigma = P_e = 4.03$ cm

The table is self-explanatory. Now, from this table

Rainfall excess = 4.03 cm.

Excess water in volume

$$= \frac{4.03}{100} \times 1200 \times 10^4 \text{ m}^3$$

$$= 48.36 \times 10^4 \text{ m}^3.$$

Since the given distribution graph peak percentage (8.2%) is based on 10 minutes interval, therefore, we should convert it into cumecs by dividing by time in sec.

$$\therefore R_e \text{ in cumecs} = \frac{48.36 \times 10^4}{10 \times 60}; \text{ for 10 minutes} = 806 \text{ cumecs for 10 minutes}$$

Peak percentage on 10 minutes interval = 8.2%

Hence, the peak runoff rate

$$= 806 \text{ m}^3/\text{sec} \times \frac{8.2}{100} = 66.2 \text{ cumecs.} \quad \textbf{Ans.}$$

Example 7.48. *On a catchment area of 200 sq km, rainfalls of 7.5 cm, 2.0 cm, and 5.0 cm occurred on three consecutive days. The ϕ_{index} was 2.5 cm/day. The distribution graph percentages of surface runoff which extended over six days for every rainfall of such magnitudes are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph and determine the peak discharge. Neglect base flow.*

(Engg. Services, 1982)

Solution. In this question, rainfall volumes as depths, are given for 3 days. To find out effective rainfall or runoff, we should subtract the daily infiltration from the daily rainfall. It means that the effective rainfall, *i.e.* runoff volume (as depth) on three consecutive days would be :

(*i*) $|7.5 - 2.5| = 5.0$ cm on first day

(*ii*) $|2.0 - 2.5| = 0$ cm on second day

(*iii*) $|5.0 - 2.5| = 2.5$ cm on third day

Hence 5.0 cm, 0 cm, and 2.5 cm of discharge volumes are produced on three consecutive days.

The discharge pattern is governed by the distribution graph percentages, contributed by each effective rain, for successive 6 days from its occurrence. These percentages when multiplied by effective rainfall, would give the discharge produced on successive 6 days from the occurrence of each effective rainfall. These discharges when added would give us the total discharge.

The calculations are carried out in Table 7.58 which works out the runoff per day caused by all the three effective rainfalls, in cm in col. (9). These values are then converted to m^3/sec for the given catchment area of 200 sq km. The ordinates of the resultant hydrograph in cumecs are thus finally obtained in col. (10) of this table. The values are plotted to obtain the requisite discharge hydrograph, as shown in Fig. 7.116.

Table 7.58

Time interval in days	*Rain fall in cm*	*Infiltration loss in cm*	*Effective rainfall in cm col. 2 – col. (3) (+ve values only)*	*Distribution graph percentages i.e. av. distribution ratio*	*Distributed runoff for rainfall excess of*			*Runoff due to three rainfall excesses*	
					5 cm	*0 cm*	*2.5 cm*	*in cm = col (6) + col. (7) + col. (8)*	*in m^3/s = 23.148* × col. (9)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
0—1	7.5	2.5	5.0	5%	5% × 5 cm = 0.250 cm			0.250	5.78
1—2	2.0	2.5	0	15%	15% × 5 cm = 0.750 cm	5% × 0 = 0		0.750	17.36
2—3	5.0	2.5	2.5	40%	40% × 5 cm = 2.000 cm	15% × 0 = 0	5% × 2.5 cm = 0.125 cm	2.125	49.1
3—4				25%	25% × 5 cm = 1.250 cm	40% × 0 = 0	15% × 2.5 cm = 0.375 cm	1.625	37.62
4—5				10%	10% × 5 cm = 0.500 cm	25% × 0 = 0	40% × 2.5 cm = 1.000 cm	1.500	34.72
5—6				5%	5% × 5 cm = 0.250 cm	10% × 0 = 0	25% × 2.5 cm = 0.625 cm	0.875	20.25
6—7				0%	0% × 5 cm = 0.000 cm	5% × 0 = 0	10% × 2.5 cm = 0.250 cm	0.250	5.79
7—8						0% × 0 = 0	5% × 2.5 cm = 0.125 cm	0.125	2.89
8—9							0% × 2.5 cm = 0.000 cm	0	0
			Σ = 7.5 cm					Σ = 7.5 cm	

*Runoff of 1 cm (1/100 m) per day for 200 sq km area = $200 \times 10^6 \times (1/100)$ cum/day = 200×10^4 cum/day

$$= \frac{200{,}0000}{24 \times 60 \times 60} \text{ cum/sec} = \mathbf{23.148\ m^3/sec.}$$

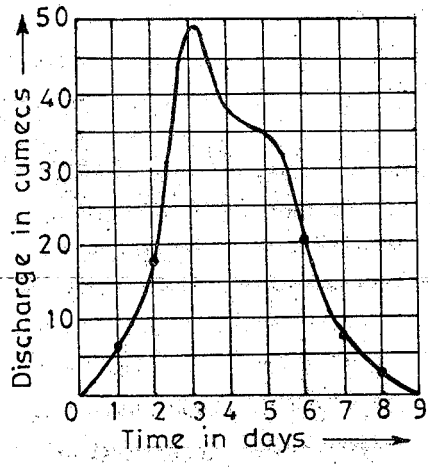


Fig. 7.116

7.47. S-Curve Hydrograph

When a unit hydrograph of a desired unit duration (t_2) is neither available nor can be developed by analysis of the rainfall runoff data, then we are left with no alternative but to somehow or the other, derive this hydrograph from the known hydrograph of some other duration (t_1). The problem is simple, when the unknown hydrograph duration is an integral multiple of the known hydrograph duration. The computations in such a case are relatively easy. Say for example, a known unit hydrograph of say 3 hr duration can be easily used to compute the unit hydrograph of 6 hr duration by superimposing the known 3 hr unit hydrograph, over the 3 hr unit hydrograph plotted with a time lag of 3 hr; and then dividing each ordinate by 2.

The problem is, however, complex when the duration of the unknown hydrograph is shorter or not an integral multiple of the duration of the known hydrograph. In such situations, a *S-hydrograph i.e.* **summation curve hydrograph,** based on the principle of superposition, is, first of all, constructed; and then used to compute the hydrograph of any desired duration (t_2).

The S-hydrograph or **S-curve** is essentially a hydrograph produced by a continuous effective rainfall at a constant rate for an indefinite period. The *S-hydrograph* is a continuously rising curve, which ultimately attains a constant value, when equilibrium discharge is reached, after the entire catchment starts contributing to runoff (Refer Fig. 7.117).

7.47.1. Determining S-curve. The *S*-curve for a t_1 hr unit hydrograph can be determined on the principle that : the *S-curve* lagged by t_1 hr when subtracted from the S-curve* gives us nothing but the unit hydrograph of t_1 hr.* In other words, when we add unit hydrograph ordinates with the ordinates of *S*-curve-lagged by t_1 hr, we get the ordinates of *S*-curve. Moreover, the initial ordinates of *S*-curve as well as the above imaginary offsetted *S*-curve, are the same as unit hydrograph ordinates. This principle

* For t_1 hr unit hydrograph of course.

can be easily used to compute *S*-curve ordinates, by simultaneously computing *S*-curve ordinates as well as offsetted *S*-curve ordinates from the known U.H. ordinates. This technique is quite simple, and *would become clear to you, when once you solve numerical example 7.48.*

7.47.2. Using S-curve for Determining Unit Hydrograph of Unknown Duration (t_2). When once the *S*-curve has been derived from the given t_1 hr unit hydrograph, we can easily use it to derive the U.H. of another duration t_2, as discussed below :

Assume that the *S*-hydrograph has been produced by continuous effective rainfall of a constant rate of q mm/hr. Then, advance or offset the position of this *S*-hydrograph (curve 1 in Fig. 7.117) for a period equal to the desired duration of t_2 hours, as shown by curve 2 in the figure. This can be called a "shifted" or "lagged" *S*-curve. The difference between the ordinates of the original *S*-curve and the shifted *S*-curve, divided by $\frac{1}{1\text{ cm}^*} q \cdot t_2$ would result in the desired unit hydrograph.

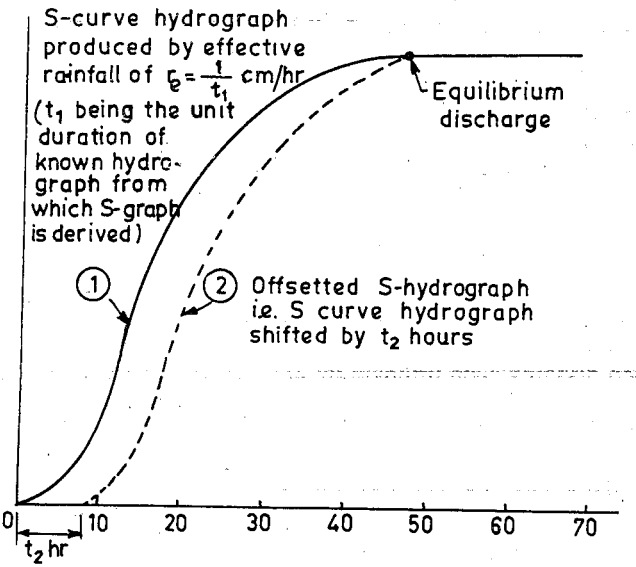


Fig. 7.117. S-curve hydrograph and its use for computing unit hydrograph of any other duration (t_2).

q can be computed easily from the 1 cm net rain contained in the U.H. of known duration (t_1), as $q = \frac{1\text{ cm}^*}{t_1\text{ hr}}$.

$$\therefore \quad q = \frac{1}{t_1}\text{ cm/hr} \qquad ...(i)$$

$$\therefore \quad q \cdot t_2 = \frac{1}{t_1} \cdot t_2 = \frac{t_2}{t_1}$$

* 1 cm will become 2.5 cm, when 1" unit hydrograph is considered instead of 1 cm hydrograph.

Hence, the difference in the ordinates of *S*-curve and shifted *S*-curve, should be divided by the factor t_2/t_1, *i.e.* **multiplied** by t_1/t_2, so as to obtain the ordinates of unit hydrograph of duration t_2.

The constant discharge of *S*-hydrograph at the time of equilibrium can also be obtained as :

Equilibrium discharge $= Q_{eq}$

$$= \left(\frac{q}{100}\right) \times (A \times 10^6) \left(\frac{1}{60 \times 60}\right) \text{m}^3/\text{sec.}$$

where, q is the effective rainfall rate in cm/hr, and
A is the catchment area in sq. km.

or $$Q_{eq} = \frac{q}{0.36} \cdot A$$

or $$Q_{eq} = \frac{1}{t_1} \frac{1}{0.36} \cdot A \qquad \left[\because \; q = \frac{1}{t_1}\right]$$

or $$\boxed{Q_{eq} = 2.78 \frac{A}{t_1}}$$ for 1 cm unit graph ...(7.114)

Also, $Q_{eq} = 6.94^* \frac{A}{t_1}$ for 2.5 cm unit graph ...(7.114 *a*)

where Q is in cumecs; A is in sq km; t is in hr.

This computation will help us in smoothening *S*-curve, when there are wild fluctuations from the terminal computed value, as discussed in example 7.49.

Example 7.49. *(a) Compute S-curve ordinates for the following 4 hr unit hydrograph of a catchment of 300 sq km area.*

Time in hr	*Unit graph ordinates in cumecs*
0	*0*
1	*6*
2	*36*
3	*66*
4	*91*
5	*106*
6	*93*
7	*79*
8	*68*
9	*58*
10	*49*
11	*41*
12	*34*
13	*27*
14	*23*
15	*17*
16	*13*
17	*9*
18	*6*
19	*3*
20	*1*
21	*0*

* $2.78 \times 2.5 = 6.94$

(b) Compute 5 hr unit hydrograph for this basin with the help of the above derived S-curve.

Solution. The given ordinates of 4 hr unit hydrograph are listed in col. (2) of table 7.59. During the first 4 hr, the unit hydrograph ordinates (col. 2) and *S*-hydrograph ordinates (col. 4) being the same, are written in col. (4).

Table 7.59

Time in hr	*4 hr unit graph ordinates cumecs*	*Imaginary offsetted S-curve (shifted by $t_1 = 4$ hr) cumecs*	*S-curve ordinates cumecs*	*S-curve lagged by $t_2 = 5$ hr cumecs*	*Difference (4) – (5)*	*Required ordinates of 5 – hr U.H. = (6) × $\frac{t_1}{t_2}$ = $\frac{4}{5}$ col (6)*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	0	—	0		0	0
1	6	—	6		6	5
2	36	—	36		36	29
3	66	—	66		66	53
4	91	0	91		91	73
5	106	6	112	0	112	90
6	93	36	129	6	123	98
7	79	66	145	36	109	87
8	68	91	159	66	93	74
9	58	112	170	91	79	63
10	49	129	178	112	66	53
11	41	145	186	129	57	46
12	34	159	193	145	48	38
13	27	170	197	159	38	30
14	23	178	201	170	31	25
15	17	186	203	178	25	20
16	13	193	206	186	20	16
17	9	197	206	193	13	11
18	6	201	207	197	10	8
19	3	203	206	201	5	4
20	1	206	207	203	4	3
21	0	207	207	206	1	1
22	0	207	207	207	0	0

The *S*-hydrograph ordinates for the second 4 hr period are now transferred to col (3) under the heading *"imaginary offsetted S-curve"*, and added to unit hydrograph ordinates in col (4); and process continued to complete column (4) and (3) both, simultaneously. The ordinates obtained in col (4) are the requisite ordinates of *S*-curve.

Checking. Now, the equilibrium discharge

$$Q_{eq} = \frac{2.78\,A}{t_1}$$

where A is in sq km and t_1 is in hr.

$$= \frac{2.78 \times 300}{4} \text{ cumecs} = 208 \text{ cumecs.}$$

Since the value of equilibrium discharge (208) tallies with the terminal value (207) obtained for *S*-hydrograph ordinate (col 4 table 7.59), no correction needs be applied to the values of *S*-hydrograph ordinates determined in col (4) table 7.59, which represents the requisite *S*-hydrograph.

Note. When equilibrium discharge approximately tallies with the computed terminal value of *S*-curve ordinates, it shows that the 4 hr period assigned to the given unit graph is precisely correct. Had there been a wild fluctuation in the terminal value as compared to the equilibrium discharge, that would have indicated an unprecise approximated rainfall duration for the unit graph. In such a case the *S*-curve, obtained in col (4), would need smoothening, as explained in example 7.50.

(*b*) To determine 5 hr (t_2 hr) unit graph from the given 4 hr (t_1 hr) *S*-curve, we will subtract from the *S*-curve (col 4) another *S*-curve lagged by t_2 hr (col 5), and then multiply the ordinates by $\frac{t_1}{t_2}\left(i.e.\ \frac{4}{5}\right)$ as done in col (6) and (7), respectively. The final ordinates determined in col (7) are the ordinates of the requisite 5 hr unit hydrograph. **Ans.**

Example 7.50. *Given below are the ordinates of a unit hydrograph, derived from a storm on a 1329 sq km catchment, and believed to have resulted from a rain of 3 hour unit duration.*

Hour	*Runoff in cumecs*
0	*0*
1	*165*
2	*547*
3	*750*
4	*580*
5	*465*
6	*352*
7	*262*
8	*195*
9	*143*
10	*97*
11	*60*
12	*33*
13	*15*
14	*7*
15	*0*

Determine the unit hydrograph of 1 hr unit duration by S-curve hydrograph.

Solution. Area enclosed under the above graph

$$= \frac{1 \times 60 \times 60}{3}\left[\begin{matrix} 0 + 2\,(547 + 585 + 352 + 195 + 97 + 33 + 7) \\ + 4\,(165 + 750 + 465 + 262 + 143 + 60 + 15) \end{matrix}\right]$$

$$= 1200\,[0 + 3632 + 7440] = 1200 \times 11{,}072 = 13.286 \times 10^6\ \text{m}^3/\text{sec}.$$

Depth of SRO on catchment

$$= \frac{13.286 \times 10^6}{1329 \times 10^6} \times 100\ \text{cm} \approx \mathbf{1\ cm}$$

It means that the given unit hydrograph contains 1 cm of rainfall, and not 2.5 cm (1 inch).

Now, the *S*-curve hydrograph is worked out in Table 7.60, as below :

The ordinates of available 3 hr unit hydrograph are listed in col. 2. During the first 3 hr, the unit hydrograph ordinates (col 2) and *S*-hydrograph ordinates (col 4) being identical, are listed in col (4). The *S*-hydrograph ordinates for the first 3 hr period are transferred to col. 3 (under the heading offsetted *S*-hydrograph), and added to unit hydrograph ordinates for the second 3 hr interval to obtain corresponding *S*-hydrograph ordinates. The process is continued to complete col (3) and col (4). A curve is now plotted between time (col 1) and the computed coordinates of S-curve (col. 4), as shown in Fig. 7.118.

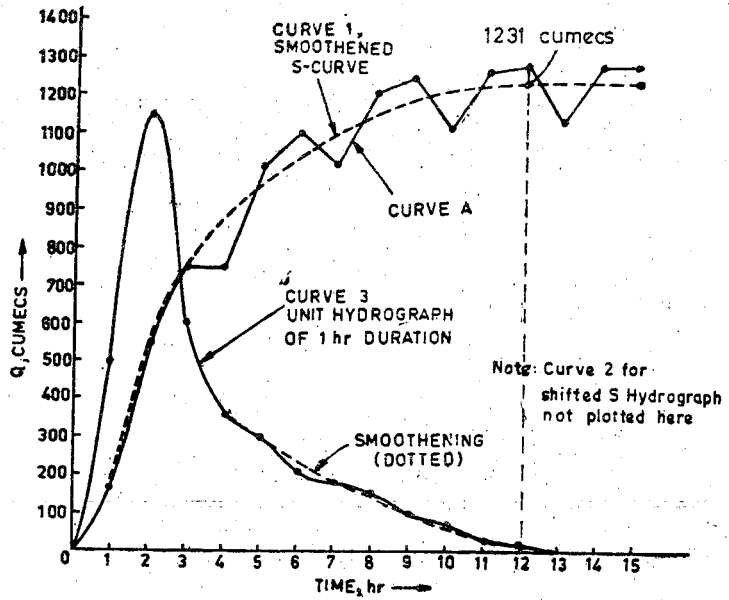


Fig. 7.118

In general, the computed ordinates of *S*-hydrograph will not plot a smooth curve (curve A); and a smooth curve is now drawn (curve 1 – dotted), by considering equilibrium discharge of $2.78\frac{A}{t_1} = 1231$ cumecs, as having reached at time equal to 15—3 (one unit duration) = 12 hr. The ordinates of smooth *S*-curve are now read out from curve 1 and listed in col. (5). **Note :** The periodic up and down variation, known as *hunting* or *waving* in the *S*-hydrograph ordinates, is due to non-uniform distribution of effective rainfall, whereas it is assumed uniform in the computations.

Now to obtain the unit hydrograph of any other duration t_2 (1 hr in this case), two *S*-hydrographs are spaced with their initial points separated by a period equal to the desired unit duration of 1 hour. The difference between their ordinates is then multiplied by $\frac{t_1}{t_2} = \frac{3}{1}$ to obtain the desired hydrograph.

Table 7.60

Time in hr	Computation of S-hydrograph from known unit hydrograph of 3 hr duration				Computation of 1 hr unit hydrograph from S-hydrograph			
	Ordinates of $t_1 = 3$ hr unit hydrograph in cumecs	Imaginary offsetted S-hydrograph (offsetted by 3 hr) ordinates in cumecs	S-hydrograph ordinates for effective rain of 1 cm per 3 hr intensity in cumecs	Smoothened S-hydrograph ordinates for effective rain of 1 cm per 3 hr ; read out from curve 1 of Fig. 7.118 in cumecs	Ordinates of S-hydrograph of col. 5 shifted by $t_2 = 1$ hr in cumecs	Diff. of col. (5) and (6) in cumecs	Ordinates of 1 hr unit hydrograph $= \frac{t_1}{t_2} \times$ col (7) $= 3 \times$ col. 7 in cumecs	Ordinates of $t_2 = 1$ hr unit hydrograph (smoothened) in cumecs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	165	–	165	165	–	165	495	495
2	547	–	547	547	165	382	1146	1146
3	750	–	750	750	547	203	609	609
4	580	165	745	870	750	120	360	360
5	465	547	1012	970	870	100	300	300
6	352	750	1102	1040	970	70	210	240
7	262	745	1017	1100	1040	60	180	180
8	195	1012	1207	1150	1100	50	150	140
9	143	1102	1245	1185	1150	35	105	105
10	97	1017	1114	1210	1185	25	75	60
11	60	1207	1267	1222	1210	10	30	30
12	33	1245	1278	1231*	1222	9	27	20
13	15	1114	1129	1231	1231	0	0	0
14	7	1267	1274	1231	1231	—	—	—
15	0	1278	1278	1231	1231	—	—	—

*Equilibrium discharge of 1 cm/3 hr effective rainfall over 1329 sq km catchment.

$$= 2.78 \frac{A}{t_1} = \frac{2.78 \times 1329}{3} = 1231 \text{ cumecs.}$$

The computation of 1-hr unit hydrograph from the smooth *S*-curve derived above, is illustrated in col. 6 to 9 of Table 7.60. The computed 1 hr-unit hydrograph may also require smoothening as done in Fig. 7.118. These smoothened ordinates of the desired hydrograph are tabulated in col. (9) **Ans.**

Example 7.51. *The ordinates of a unit hydrograph of a 6 hr. unit duration for a catchment of 542 sq km are given below :*

Time from beginning of rainfall in hr.	*Ordinates of unit hydrograph in cumecs*
0	*0*
6	*20*
12	*50*
18	*150*
24	*120*
30	*90*
36	*70*
42	*50*
48	*30*
54	*20*
60	*10*
66	*0*

Compute the ordinates of a 3 hr. unit hydrograph from this 6 hr. unit hydrograph, using S-curve method.

Solution. First of all, we will ascertain whether the given unit hydrograph contains 1 cm or 1 inch of effective rainfall. So using Simpson's formula, we have

$$\text{Area under graph} = \frac{H}{3}\left[\begin{matrix}(\text{First} + \text{Last ordinate}) + 2\ \text{Odd}\\ \text{ordinates} + 4\ \text{Even ordinates}\end{matrix}\right]$$

$$= \frac{6\text{ hr}}{3}\left[\begin{matrix}(0+0) + 2\,(50+120+70+30+10)\\ +\,4\,(20+150+90+50+20)\end{matrix}\right]$$

$$= 2\text{ hr}\,[2\times 280 + 4\times 330] = 2\text{ hr}\,[560 + 1320]\text{(cumecs)}$$

$$= 2\times 60\times 60\ \text{(secs)}\ [1880]\ (\text{m}^3/\text{sec.})$$

$$= 13.536\times 10^6\ \text{m}^3$$

$\therefore$ Depth of water over the basin

$$= \left[\frac{13.536\times 10^6\times 100}{542\times 10^6}\right]\text{cm} \approx 2.5\text{ cm}$$

Hence, *the given unit graph contains 1 inch of rainfall.*

Now, from the given 6 hr. unit hydrograph, we will derive an *S*-curve of 2.5 cm/6 hr effective rainfall intensity.

The ordinates of 6 hr. unit hydrograph are listed in col. (2) of Table 7.61. During the first 6 hours, the unit hydrograph ordinates [col. (2)] and *S*-curve ordinates (col. 4) are the same. The *S*-curve ordinates for the second 6 hr. period are transferred to col. 3

under the heading 'imaginary offsetted *S*-curve' and added to unit hydrograph ordinates; and process continued to complete col. (4) and (3) both simultaneously. Now, the equilibrium discharge at a time equal to the base of the unit hydrograph minus the unit duration is worked out as below :

Table 7.61

Time in hr from start	*Computation of S-curve from known 6 hr unit hydrograph*		
	Ordinates of 6 hr unit hydrograph in cumecs	*Imaginary off setted S-curve (shifted by 6 hr) ordinates in cumecs*	*S-curve ordinates for effective rain of 2.5 cm per 6 hr intensity in cumecs*
(1)	*(2)*	*(3)*	*(4)*
0	0	—	0
6	20	0	20
12	50	20	70
18	150	70	220
24	120	220	340
30	90	340	430
36	70	430	500
42	50	500	550
48	30	550	580
54	20	580	600
60	10	600	610
66	0	610	610*

$$Q_{eq} = 6.94 \cdot \frac{A}{t_1}. \qquad \text{(for "1" } U \text{ graph)}$$

$$= 6.94 \times \frac{542}{6} \text{ cumecs} = 626 \text{ cumecs.}$$

Since Q_{eq} (626 cumecs) approximately tallies with the terminal value (610) obtained in col. (4), we may ignore this minor variation, and hence there is no need to further smoothen the obtained S-curve ordinates.

Now, we have to find out another 'shifted *S*-curve', lagging by $t_2 = 3$ hr, and then subtract it from the *S*-curve values of col. (4). But since the given ordinates are at 6 hr. interval, it is necessary to compute them at 3 hr interval by first plotting the *S*-curve on a graph (Fig. 7.119)**.

The ordinates of *S*-curve, plotted with values of col. (4) table 7.61 in Fig. 7.119, are used to read out its ordinates at 3 hr interval, and these are stipulated in col. (2) of Table 7.62. The *S*-hydrograph values shifted by 3 hr are written in col. (3) at a lag of 3 hr.

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* Should be equal to the equilibrium discharge, to avoid smoothening of S-curve.

** *Alternatively,* shifted S-curve itself may be plotted and its ordinates at 6 hr. are readout.

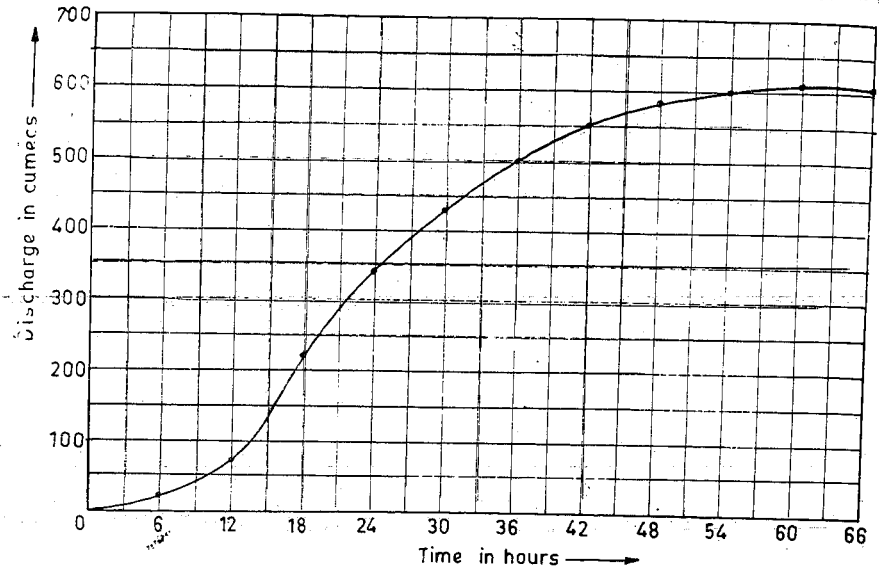


Fig. 7.119

Table 7.62

Time in hr. (1)	*S-curve for U.H. of* $t_1 = 6$ *hr.* (2)	*S-curve lagged by* $t_2 = 3$ *hr.* (3)	*Difference col. (2) – col. (3)* (4)	*Ordinates of U.H. of 3 hr* $= Col.\ (4) \times \frac{t_1}{t_2}$ *(i.e.* $\frac{6}{3} = 2$*)* (5)
0	0	—	0	0
3	8*	0	8	16
6	20	8	12	24
9	40*	20	20	40
12	70	40	30	60
15	130*	70	60	120
18	220	130	90	180
21	282*	220	62	124
24	340	282	58	116
27	390*	340	50	100
30	430	390	40	80
33	468*	430	38	76
36	500	468	32	64
39	527*	500	27	54
42	548	527	21	42
45	565*	548	17	34
48	580	565	15	30
51	592*	580	12	24
54	600	592	8	16
57	606*	600	6	12
60	610	606	4	8
63	610*	610	0	0
66	610	610	0	0

* Alternate intermediate values read from Fig. 7.119.

Difference of these two *S*-curve ordinates are worked out in col. (4), which is finally multiplied by $\frac{t_1}{t_2} = \frac{6}{3} = 2$ to obtain the requisite U.H. in col. (5). **Ans.**

Example 7.52. *A 2 hr unit hydrograph in a rather steep catchment is given below:*

Time, hr	*0*	*2*	*4*	*6*	*8*	*10*	*12*
Discharge 100 m^3/s	*0*	*0.54*	*1.75*	*1.27*	*0.58*	*0.25*	*0*

Compute the 1 hr unit hydrograph for the catchment. (*Engg. Services, 1989*)

Solution. We will first determine *S*-curve ordinates, as shown in col. 4 of Table 7.63. We have now to stipulate ordinates of *S*-curve shifted by 1 hr; but since the ordinates given are at 2 hr interval, we will have to use a graph to determine intermediate values at 1 hr interval, for the *S*-curve.

Table 7.63

Time in hr.	*Discharge in cumecs*	*Imaginary S-curve in cumecs*	*S-curve for U.H. of t_1 = 2 hr in cumecs*	*S-curve with hourly values (with interpolation from graph of Fig. 7.120) in cumecs*	*Offsetted S-curve by t_2 = 1 hr in cumecs*	*Col (5) – col. (6) in cumecs*	*Ordinates of t_2 = 1 hr U.H. =Col. (7) × (2) $\left(\because \frac{t_1}{t_2} = 2\right)$ in cumecs*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
0	0		0	0		0	0
1				25*	0	25	50
2	54	0	54	**54**	25	29	58
3				118*	54	64	128
4	175	54	229	**229**	118	111	**222** (Peak)
5				303	229	74	148
6	127	229	356	**356**	303	53	106
7				390	356	34	68
8	58	356	414	**414**	390	24	48
9				430	414	16	32
10	25	414	439	**439**	430	9	18
11				439	439	0	0
12	0	439	439	**439**	439	0	0

Hence, the *S*-curve (col. 4) is plotted in Fig. 7.120, and intermediate ordinates at odd hrs are read out from this figure, and all these ordinates are rewritten in col. (5). The 1 hr lagged *S*-curve ordinates are then written in col. (6).

* Intermediate values read from Fig. 7.120. If graph sheet is not available, arithmetic average values may be used.

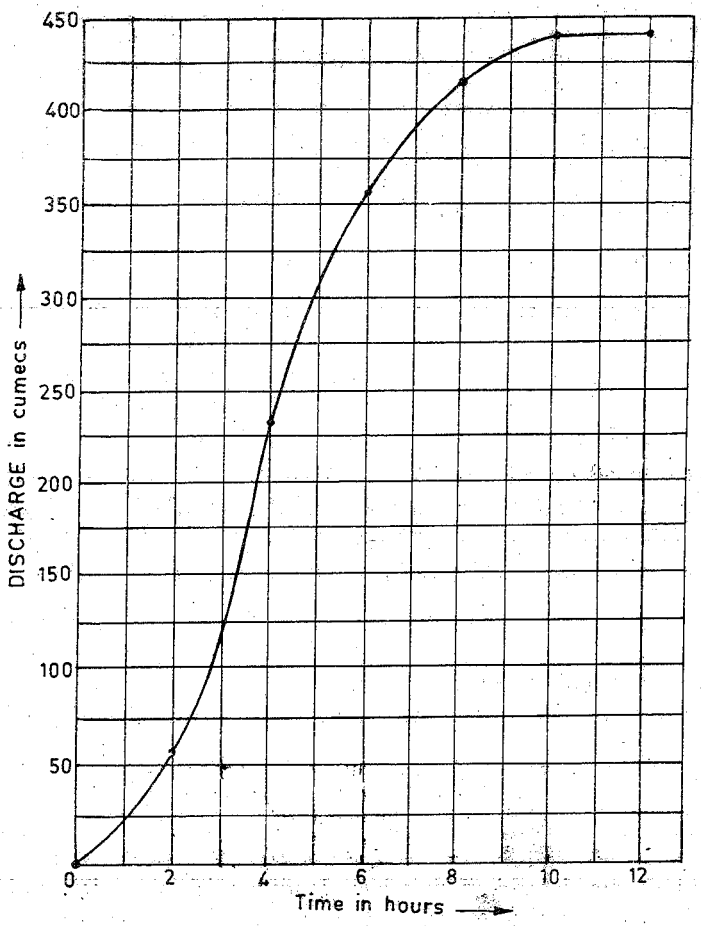


Fig. 7.120

Difference of col. (5)— col. (6) is written in col. (7), and finally col. (7) is multiplied by $\frac{t_1}{t_2} = \frac{2}{1} = 2$, to obtain the requisite U.H. ordinates in col. (8). **Ans.**

Note. Smoothening of *S*-curve has not been carried out, firstly because the question was only of 25 marks, and secondly because catchment area was not given, and yet when we approximate it on Simpson's rule, and on assuming that the given U.H. is of 1 cm, Q_{eq} works out to only 430* cumecs, which almost tallies with the computed terminal value of 439.

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* Vol. of water in U.H.

$$\frac{1}{100}\text{ m depth} \times Am^2 = \frac{2\text{ hr} \times 60 \times 60}{3}[0 + 2(175 + 58) + 4(54 + 127 + 125)]$$

$$= 2400 \times 1290 = 309.6 \times 10^4\text{ m}^2$$

or $A = 309.6$ sq. km

Using $Q_{eq} = 2.78\frac{A}{t_1} = \frac{2.78 \times 309.6}{2} = 430$ cumecs.

ESTIMATION OF FLOOD DISCHARGE

7.48. Definition and Causes of Floods

A flood may be defined as an overflow coming from some river or from some other body of water. A river may get flooded due to excessive rainfall or excessive melting of snow or due to some other form of ice obstruction in the form of jams. *Whenever the water overflows the banks of the river, the river is said to be flooded.* In fact, the banks of the stream may vary in height throughout its course, and hence, there is no definite stage at which the whole river is about to overflow. However, on important rivers, arbitrary representative *warning* and *danger levels* are generally assigned at important locations, and are called **flood stages.**

Apart from the overflow of rivers, the floods may be caused by the failure of some dam, with a sudden release of huge amounts of water, causing considerable damage to life and property.

A very interesting example of formation and failure of a reservoir occurred in India in 1894. In Sept. 1893, the River Bireh Ganga was completely blocked by a land slide near Gohna in Garhwal. The river valley got filled up by rocky earth to a depth of about 240 metres extending for a length of about $3\frac{1}{2}$ km, thus, forming a kind of a dam. The discharge from a drainage area of about 230 sq kilometres was held back against this obstruction for nearly an year. The lake formed on the upstream side was surveyed by the worried Indian Corps of Engineers. After investigating the capacity of the lake, it was forecasted by them, that its probable capacity of 46.6 million cubic metres will be filled up by Aug. 15, 1894. In fact, the breach occurred on 25th Aug. 1894 (11.30 P.M.), and due to this breach, huge amount of water was released. So much so that at 4 P.M. of Aug. 26th, the river level had fallen by about 120 metres and by that time about 28.3 million cubic metres of outflow had occurred. The maximum water rises caused by this extraordinary flood were observed. Starting from a rise of about 80 metres in the near vicinity, the water-spread reached a distance of about 20 km. in about 25 minutes, where the rise was about 50 metres; and at a distance of about 240 kilometres, the rise was about 3.4 metres in about 12 hr.

Thanks to those engineers who forecasted this flood very correctly and efficiently, and arrangements were so well made in advance that no lives were lost in this unique flood of history.

7.49. The Design Flood and its Importance

Whenever an important structure is to be constructed on a river or in the vicinity of a river, it must be properly planned and designed keeping in view the damage to which it is susceptible and the catastrophe which it is going to create in the event of its failure. Hence, while designing any important engineering structure, provision must be made for the flood that is likely to occur during the life time of that particular structure. For example, the dams must have sufficient spillway capacity and adequate protection against the scour at the toe, bridges must have sufficient waterway openings, flood walls and levees must be high enough to avoid their overtopping, and so on.

Therefore, while designing structures, we have to think of a flood-value against which these structures can be designed to be safe. *We can neither choose a very high value nor we can choose a very low value. Because a very high value chosen for the design, will make the design very costly causing unnecessary investment, since such a high valued flood may never occur during the life time of the structure. In the same way,*

a very low value chosen for the design, if exceeded during the life time of the structure, will result in the failure of the structure causing much more damage than what it would have caused in the absence of the structure. Depending upon the magnitude, the floods can be classified into the following three classes.

(*i*) **Ordinary floods** are the floods that are sure to be equalled in magnitude once or more times in the estimated life of the project.

(*ii*) Floods exceeding the "ordinary floods" and equal to or less than the "Standard project flood" for the drainage area involved. The **Standard project flood** (SPF) is the flood that is likely to be exceeded in magnitude only at *rare occasions,* and thus, constitutes a standard for the design of structures, that would provide enough flood protection. Although, the standard project flood is of such a high magnitude, as is to be considered exceptionally infrequent, yet it is reasonably certain to occur on the drainage basin concerned. The standard project flood is generally much less than the **probable maximum flood** (PMF) *that might occur under the worst meteorological and hydrological conditions.*[*]

(*iii*) This includes floods ranging in magnitude between *standard project flood* and *maximum probable flood.*

As it is economically and practically unfeasible to provide flood control measures against all floods upto PMF value, it becomes necessary to decide how much money can be invested for flood protection, and also to decide the magnitude and frequency of the 'design flood', *i.e.* the flood against which the structure is to be designed safe. In making this decision, both the tangible and intangible benefits likely to be derived from the subsequent flood-protection must be considered.

It must be emphasised that the provision of a partial degree of protection for industrial and residential areas is worse than no provision at all. Because in the event of such a provision at a place, people will feel safe against floods, and the place will attract more people, and industries. But if the provision gets exceeded, it will result in a huge loss of life and property. All such factors must be considered before arriving at a particular design flood value against which the project is to be designed safe.

7.50. Estimating Design Flood and Flood Flows

No method is available by which the exact amount and intensity of the rainfall in any assigned future period can be predicted precisely. How-so-ever elaborate the base data may be, the rainfall and the expected flood flow cannot be predicted with absolute certainty and precision. The expected flood and its consequent damages can only be judged and approximated, and hence, while designing flood protection works, the *discretion and judgement of the design engineers is of utmost importance.*

Various methods have been used for estimating floods. Some of them are based on the characteristics of the drainage basin, and others are based on the theory of probabilities applied to the previous known flood data; and lastly, still others are based on a study of the rainfall and runoff data.

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* Typicaly, the SPF is about 40 to 60% of the PMF for a given drainage basin. SPF is that particular flood which can occur in the basin but is extremely unlikely to occur. This value cannot be so precisely defined as the probable maximum, yet it may be taken generally, as the largest flood corresponding to the largest storm which has occurred in the region of the basin during the period of weather records. It is not maximised for most critical weather conditions, as is done for determining the probable maximum flood. It may sometimes be transposed from an adjacent region to the watershed under consideration, if sufficient records are not available for the concerned area.

Several of these methods are often employed together, and a value of the design flood is chosen, so as to suit an individual problem. It has been emphasised earlier that a compromise has to be sought between the cost of flood control works, and the benefits likely to be derived from such works, simultaneously keeping an eye on human life that is likely to be lost in the absence of such works or due to inadequate works. But plainly speaking, engineers usually have a tendency to predict too high values to avoid failures of the structures in future. They, may therefore, foresee such heavy floods which may not occur at all for years and years together. Say example, while designing a dam reservoir, an engineer may foresee a design flood which may occur once in 5000 years, while the total life of the reservoir may be only 200 to 300 years or so. This, in fact, may raise the cost of construction considerably, which is an extra useless provision, and is thus, a wastage of public money. Hence, while estimating a design flood, one must not forget the economical aspect of the flood value chosen for the design.

Various methods, which are generally used for determining flood flows, can be classified into the following four classes:

(1) *determination by means of Empirical formulae ;*

(2) *determination by Envelope curves ;*

(3) *determination by Statistical or Probability methods;* and

(4) *determination by Unit hydrograph method.*

All these methods have their individual limitations, merits and demerits, as will be discussed slightly late, and the selection of a particular method or a combination of some of them depends purely upon the requirements of an individual project. These methods are described here in details.

7.50.1. Determination of Peak Flood Discharge by means of Empirical Formulae. Many formulae have been devised for the purpose of estimating flood flows. They can be safely applied to the areas for which they were specifically derived, but some of them have found a general existence. *But these formulae must be used with great prudence, and must never be used unless their origin has been investigated.* No particular formula will give precise results for all the places. This is because of the fact that the magnitude of the flood of a given frequency depends upon fifteen to twenty factors, and no formula involves all these variables. Hence, a formula involving only two or three variables in place of twenty variables, cannot be expected to give generalised precise results. Some of these important formulae are given below.

(*a*) *The formulas involving drainage area only.*

(1) **Dicken's formula (1885).** Dicken made the first attempt in India to derive a general formula for determining maximum flood on the basis of studies conducted for determining waterways capacities for bridges. His formula is generally used for the catchments of northern and central parts of India. It states that

$$Q_p = CA^{3/4} \qquad ...(7.115)$$

where Q_p = High flood or peak discharge in cumec.

A = Catchment area in sq. kilometre.

C = A constant, depending upon all those fifteen to twenty factors, which affect high flood discharge.

The value of C must be ascertained for each catchment depending upon the nature of the catchment and the intensity of rainfall. The value of C will be more for hilly catchments as compared to plain areas even for the same region. Secondly, for the same

type of catchment, greater is the rainfall, greater will be the value of C and *vice-versa*. The values of C for various locations in India are shown in Fig. 7.121 ; and zonewise values are also given in Table 7.64.

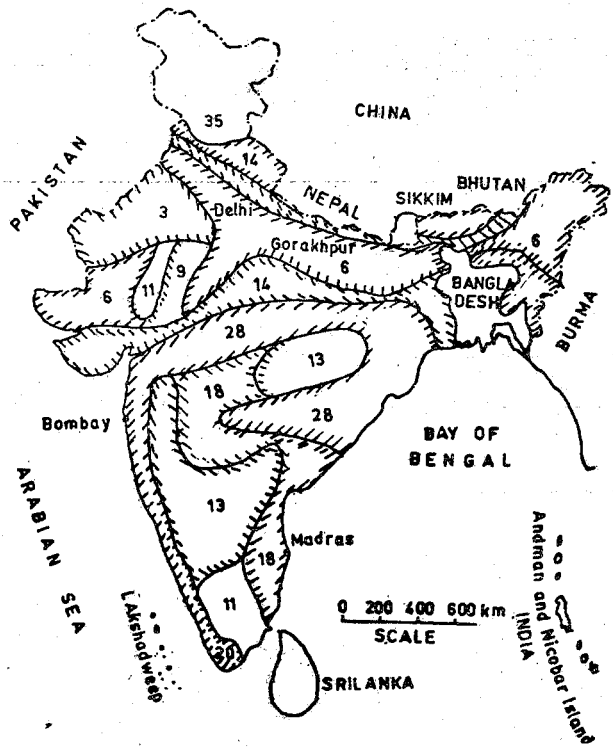


Fig. 7.121. Values of C to be used in Dicken's formula for different parts of India.

Table 7.64. Zonal Values of C for Dicken's Formula

Region	*Value of C in Dicken's formula*
North Indian plains	6
Northern Indian hilly regions	11 to 14
Central India	14 to 28
Coastal Andhra and Orissa	22 to 28
Western Ghats of India	26 to 28

(2) **Ryve's formula (1884).** This formula was originally developed for Tamil Nadu region and is in use in Tamil Nadu, and parts of Karnataka and Andhra Pradesh of South India. This formula is almost similar to that of *Dicken's*, and the only difference is in the values of the constants. It states that

$$Q_p = C_1 A^{2/3} \text{ in cumec} \qquad ...(7.116)$$

where Q_p, C_1 and A have the same meaning as in Dicken's formula.

The average value of C_1 to be used is 6.8, with lesser value for flat catchments and more for hilly catchments. Different values of C_1 must be ascertained for different catchments. Representative values are given in Table 7.65.

Table 7.65. Values of coustant C_1 to be used in Ryve's Formula

Location of Catchment	*Value of C_1 in Ryve's formula*
Areas within 80 km from the east coast	6.8
Areas within 80—160 km from the east coast	8.5
Limited areas near hills	10.2
Actual observed values	up to 40

(3) **Inglis' formula (1930).** This formula is based on flood data of catchments in western ghats in Maharashtra. It is, thus, applicable to fan-shaped catchments in old Bombay State, and is expressed as

$$Q_p = \frac{123\,A}{\sqrt{A + 10.4}} \text{ in cumec} \cong 123\sqrt{A} \qquad ...(7.117)$$

where A = The area of catchment in sq kilometres.

(4) **Nawab Jung Bahadur formula.** This formula has been derived for *Hyderabad Deccan catchments,* and states that

$$Q_p = C \cdot A_1^{\left[0.93 - \left(\frac{1}{14}\right)\log A_1\right]} \text{ in cumec} \qquad ...(7.118)$$

where C = a constant, having a value of about 49 for south India, and 60 for North India.

A_1 = Area in square miles

= 0.39 A, where A is the area in sq km.

This formula is widely used in Maharashtra for designs in small catchments.

(4) **W.P. Creager's formula.** This formula was evolved by creager from the study of flood peaks in U.S.A, and states that

$$Q_p = C \cdot A_1^{0.894 A_1^{-0.048}} \text{ in cumec.} \qquad ...(7.119)$$

where C = 40 to 130; lower values for ordinary floods, and higher values for intense and acute floods.

A_1 = catchment area in sq. miles.

= 0.39 A, where A is the catchment area in sq. km.

Graphs between discharge and drainage area were plotted by Creager in U.S.A., on the basis of which, this formula was worked out. Similar curves were plotted in India by Shri Kanwar Sain for North Indian as well as for South Indian rivers. These curves are shown in Fig. 7.122, and can be used if desired.

(6) **Jarvis formula.** It states that

$$Q_p = C\sqrt{A} \text{ in cumec.} \qquad ...(7.120)$$

where C varies between 1.77 as minimum and 177 as maximum. Limiting or 100 per cent chance floods are given by the value of C of 177.

(7) **Modified Myer's formula.** It states that

$$Q_p = 177\,p\sqrt{A} \text{ in cumec.} \qquad ...(7.121)$$

where p has a value of unity for stream that has the greatest flood flow of the area. For any other stream, p is the fraction that the flood flow of that stream is of the maximum given above. For different streams, values of p vary from 0.002 to 1.0; usually taken as 1.

(*d*) *Formulas Involving Drainage Area and its Shape*

The previous formulae were based only upon the more important variable *i.e.* the area of the drainage basin, but here are presented certain formulae making use of the area as well as the shape of the basin.

(8) **Dredge or Burge's formula.** It is based on Indian records, and states that

$$Q_p = 19.6 \frac{A}{L^{2/3}} \text{ in cumecs.} \qquad ...(7.122)$$

where A and Q_p have the same meaning, and L is the length of the drainage basin in kilometres.

If B is the average width of the basin in km; then

$$A = BL$$

The above formula can then be written as,

$$Q_p = \frac{19.6\,(B.L)}{L^{2/3}}$$

$$= 19.6\, B{\cdot}L^{1/3} \qquad ...(7.123)$$

(9) **Pettis formula.** It was originated for Northern United States, Ohio to Connecticut, by Pettis, and it states that

$$Q_p = C\,(P{\cdot}B)^{5/4} \text{ cumecs} \qquad ...(7.124)$$

where P = Probable 100 year maximum 1 day rainfall in cm.

B = Average width of the basin in kilometres

C = A constant which is equal to 1.5 for humid areas and 0.2 for desert areas.

This formula is generally applicable to areas ranging from 1,600 to 16,000 square km, with no storage effect and fairly uniform width. If the area is wider at the lower end, the formula gives too small results, and if it is wider at its upper end, the results obtained are too large. A correction of 10 to 13 per cent may be required for width variations.

(*c*) *Formulae Involving Total Runoff and Drainage Area*

(10) **Boston society of Civil Engineers formula.** The committee on floods of *Boston Society of Civil Engineers* made a study of floods, and based on the new England flood of November 1927, it stated that

$$q_p = 0.0056 \times \frac{D}{t} \qquad ...(7.125)$$

where q_p = Peak flow in cumecs/square km.

D = Total depth of flood runoff on the basin in cm.

t = Total flood period in hours or the base of the flood hydrograph.

This formula was evolved by assuming the flood hydrograph as a triangle.

This formula was found to give reasonably satisfactory results when checked for numerous rivers. It was also found that the base of the hydrograph, *i.e.* t remains almost constant irrespective of the size of the flood.

Where no flood hydrographs are available, the committee, suggested

$$q_p = \frac{C_F \cdot D}{\sqrt{A}} \qquad ...(7.126)$$

or

$$Q_p = \frac{C_F \cdot D \cdot A}{\sqrt{A}} = C_F \cdot D \cdot \sqrt{A} \qquad ...(7.127)$$

where C_F = Coefficient of the stream or flood characteristics, generally varying from 0.7 to 3.5, but upto 7 in mountains and less than 0.7 for very flat streams.

A = drainage area in sq. km.

Q_p = Peak flow in cumecs.

For new England, value of D generally varies from 7.5 to 5 cm for occasional floods to rare floods, and is not over 20 cm even for maximum floods.

(d) Formulae Involving Rainfall Intensity and Drainage Area.

(11) **Rational formula.** This is the oldest and best known formula that has been extensively used for design of storm-sewers, and was earlier stated by equation (7.105), as:

$$Q_p = \frac{1}{36}(K \cdot p_c \cdot A)$$

where K = The percentage of rainfall that becomes surface runoff, called runoff coefficient.

p_c = Critical rainfall intensity in cm/hr, generally taken as the average rate during the time of concentration T_c and can be determined from the Intensity-duration curve for the given frequency flood.

A = Catchment area in hectares.

It can also be shown that the formula suggested by Boston Society of Engineers vide eqn. (7.127) is nothing but a reduced form of Rational formula, as shown below :

The total amount of rain fallen during a time T_c is given by

$$P = p_c \cdot T_c \text{ (in cm)}$$

Therefore, the runoff resulting from this rain

$$= K \cdot p_c \cdot T_c \text{ (in cm)} = D.$$

$\therefore K$ is the percentage of the rainfall that becomes surface runoff.

or $$D = K \cdot p_c \cdot T_c$$

or $$K \cdot p_c = \frac{D}{T_c}$$

But according to the Rational formula stated above, the maximum flood flow Q_p is given by

$$Q_p = \frac{1}{36} \cdot (K \cdot p_c) \cdot A$$

Putting $K \cdot p_c = \frac{D}{T_c}$, we get

$$Q_p = \frac{1}{36} \cdot \frac{D}{T_c} A.$$

For smaller drainage areas, T_c will vary directly with a linear dimension, or as square root of area.

$$\therefore \quad T_c \propto \sqrt{A}$$

$$\therefore \quad Q_p \propto \frac{D}{\sqrt{A}} \cdot A$$

or $$Q_p \propto D\sqrt{A}$$

or $$Q_p = C_F \cdot D \cdot \sqrt{A}$$

This is the same formula as was suggested by Boston Society for Civil Engineers, and has been stated earlier vide eqn. (7.127).

The Rational formula assumes a direct relationship between the rainfall rate and the runoff rate, while that is not so in actual practice. In other words, this formula ignores the fact that for any watershed, runoff occurs in the form of a typical hydrograph as explained earlier. This formula has already been discussed in details in article 7.44.

(*e*) *Formulas involving Drainage Area and Flood Frequency.*

(12) **Fuller's formula (1914).** This formula was derived for catchments of U.S.A., and is a typical one of this kind. It states that

$$Q_{T_p} = C_f A^{0.8} \left[1 + 0.8 \log T\right] \qquad ...(7.128)$$

where Q_{T_p} = Peak 24-hour flood with a frequency of T years, in m^3/s.

A = Catchment area in km^2.

C_f = Fuller's constant with values varying between 0.18 to 1.88

7.50.2. Determination of Peak Flood Discharge by Using Envelope Curves. *Kanwar Sain* and *Karpov* collected the data on Indian rivers, and have drawn two envelope curves, as shown in Fig. 7.122. One curve suits basins of South India, and the other suits the basins of Northern and Central India. Since in these curves, the flood discharge has been connected with the drainage area only, and other basin characteristics have been ignored, the results obtained will not be precise. However, they can be used for preliminary guidance for determining peak flood discharges, and are better than the empirical formulas, in the sense that here the selection of coefficients on the basis of judgement is not required, as is required in empirical formulas.

The inherent limitation of these curves lies in the fact that they are based on past records available upto the time such curves are drawn. Such curves should therefore, be revised from time to time, as more and more data become available. Moreover, the flood peak obtained from such a curve will tell us the maximum flood which occurred in the available data, and cannot tell us anything about the future possibilities. The peak value

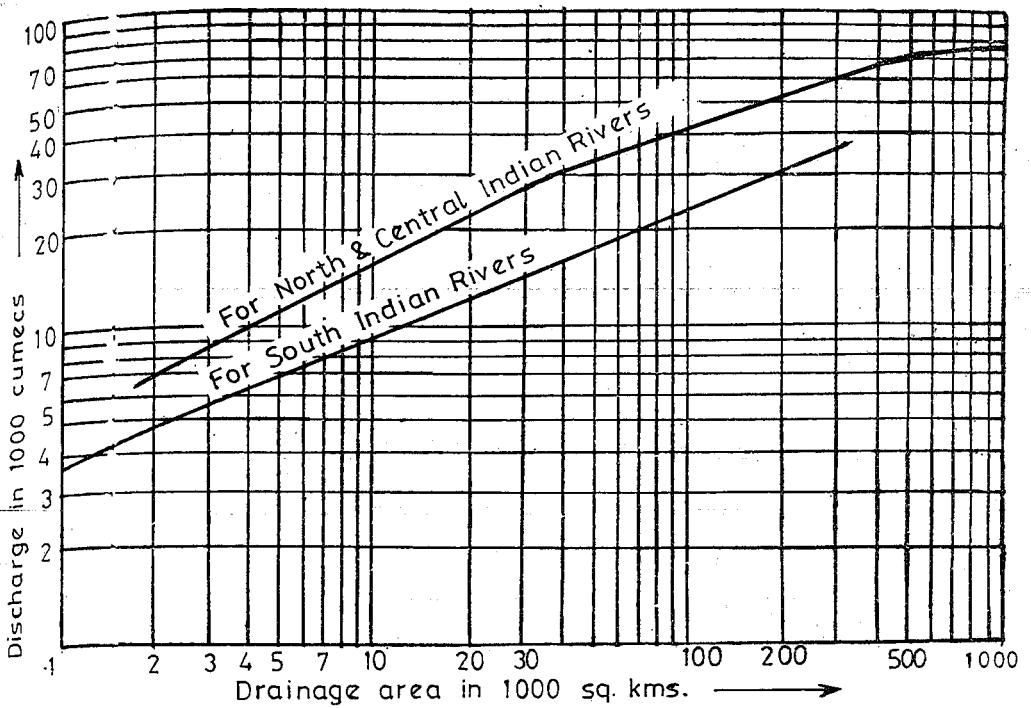


Fig. 7.122. Envelope curves.

obtained from this curve cannot, therefore, be relied upon, as it will give lesser value for probable maximum flood, and a higher value for the design of minor structures. They can, therefore, only be used for preliminary guidance, and have been used as such in certain dam projects, such as Nagarjunsagar (Andhra Pradesh), Matatila (U.P.), Deonur project (Karnataka), etc.

7.50.3. Determination of Discharges for Different Frequency Floods by Using Statistical or Probability Methods. In these methods, the predictions for the future floods are made on the basis of the available records of the past floods. These methods can be safely used to determine the maximum flood that is expected on a river with a given frequency, if sufficient past records are available. Secondly, *the predictions will be precise only if there has occurred no appreciable change in the regime of the river during or after the period of records.*

These **probability methods** are however, unable to give precise results, where lesser number of past records are available. For the success of any probability method, sufficient past records must, therefore, be made available.

These methods were of great importance, and were almost exclusively used before the advent of Unit Hydrograph Theory, although of course, their use still continues on a large scale.

Before these probability methods are discussed in greater details, let us first define the term *'chance flood'* or *'chance percentage'*.

Chance flood. If a flood of a given magnitude occurs with an average frequency of 100 years, then there exists $\frac{1}{100} \times 100 = 1$ per cent chance for this flood to occur, and such a flood is generally called 1 per cent chance flood. Because this is a flood which can occur once in 100 years in any year, and this meaning of it, becomes quite evident by calling it as 1 per cent chance flood. Similarly, a flood having an average frequency of 20 years (*i.e.* a flood which is likely to be equalled or exceeded once in 20 years) can be designated as $\frac{1}{20} \times 100 = 5\%$ chance flood. This method of designating floods by the reciprocal of its frequency or recurrence interval ($\times 100$) was suggested by Hazen, and may be preferred to the traditional method of representing floods by their frequencies or recurrence intervals, because a common man can understand its meaning more clearly. "A flood having a frequency of 100 years" is generally misunderstood as a flood that will occur after 100 years. But "1 per cent chance flood" shows clearly that, on an average, it is a flood which can occur at any one time during 100 years.

Hence, the **probability of occurrence** of such a 100 year flood equalling or exceeding in a given year, would be once in 100 years, *i.e.* $\frac{1}{100} = 0.01$. This **probability of occurrence or exceedance** is generally represented by p, and would be equal to $\frac{1}{T}$.

$$\therefore \qquad p = \frac{1}{T} \text{ where } T \text{ is the recurrence interval.} \qquad ...(7.129)$$

If the probability of occurrence of an event (such as precipitation or flood) is p, then the *probability of its non- occurrence* is q, which would be equal to $1 - p$.

Hence, $$q = 1 - p \qquad ...(7.130)$$

The binomial distribution can be used to find the **probability of the event occurring r times in n successive years,** as :

$$p_{r,n} = {}^{n}C_r \cdot p^r \cdot q^{n-r}$$

or $$\boxed{p_{r,n} = \frac{\underline{/n}}{\underline{/(n-r)}\,\underline{/r}} p^r \cdot q^{n-r}} \qquad ...(7.131)$$

Say for example,

(*a*) When $r = 2$, then, the probability of occurrence/exceedance of an event twice in n successive years, would be:

$$p_{2,n} = \frac{\underline{/n}}{\underline{/(n-2)} \cdot \underline{/2}} \cdot p^2 \cdot q^{n-2} \qquad ...(7.132)$$

(*b*) When $r = 1$, then, the probability of occurrence/exceedance of an event *once* in n years would be:

$$p_{1,n} = \frac{\underline{/n}}{\underline{/(n-1)}} p^1 \cdot q^{n-1} = n \cdot pq^{n-1} \qquad ...(7.133)$$

Similarly when $r = 0$, then

$$p_{0,n} = \frac{\underline{/n}}{\underline{/n-0} \cdot \underline{/0}} \cdot p^0 \cdot q^{n-0}$$

$$= \frac{\underline{n}}{\underline{n}} q^n$$

or $$p_{0,n} = q^n \qquad ...(7.134)$$

Hence, the probability of an event *not occurring at all* in n successive years would be equal to q^n, which is equal to $(1-p)^n$. Also, the probability of an event *occurring at least once* in n successive years (R) would evidently be equal to $1-q^n$ or $[1-(1-p)^n]$

or $$\boxed{R = 1 - q^n = [1-(1-p)^n]} \qquad ...(7.135)$$

This probability is called risk and hence represented by R.

Example 7.53. *A flood of a certain magnitude has a return period of 25 years: (a) What is its probability of exceedance? (b) What is the probability that this flood may occur in the next 12 years.* *(U.P.S.C. Engg. Services, 1983)*

Solution. (*a*) $T = 25$ years

$$p = \text{exceedance probability} = \frac{1}{T} = \frac{1}{25} = 0.04 \textbf{ Ans.}$$

(*b*) The probability of *non-occurrence* of a flood in next n successive years is given by equation

$$p_{0,n} = q^n = (1-p)^n$$
$$= [1-0.04]^{12} \qquad (\because \ n = 12 \text{ given})$$
$$= (0.96)^{12} = 0.613.$$

$\therefore$ Probability of *occurrence* of this flood at least once in next 12 years.

$$R = 1 - 0.613 = \textbf{0.387. Ans.}$$

Example 7.54. *On the basis of isopluvial maps, the 50 year-24 hr maximum rainfall at Bangalore is found to be 16 cm. Determine the probability of 24 hr rainfall of magnitude equal to or greater than 16 cm occurring at Bangalore:*

(*a*) *at least once in 10 successive years;*

(*b*) *two times in 10 successive years; and*

(*c*) *once in 10 successive years.*

Solution. Frequency of rainfall

$$= T = 50 \text{ years}$$

$$p = \text{exceedance probability}$$
$$= \frac{1}{T} = \frac{1}{50} = \textbf{0.02}$$

(*a*) Probability of equalling or exceeding *at least once* in n successive years, is given by Eqn. (7.135), as:

$$R = [1-(1-p)^n]$$

or $$R = 1 - (1-0.02)^{10}; \qquad \because \ n = 10 \text{ years (given)}$$
$$= 1 - (0.98)^{10} = 1 - 0.817 = \textbf{0.183} \quad \textbf{Ans.}$$

(*b*) Probability of occurrence twice in *n* years is given by eq. (7.132), as:

$$p_{2,n} = \frac{n!}{(n-2)! \cdot 2!} p^2 \cdot q^{n-2}$$

or
$$p_{2,10} = \frac{10!}{8! \cdot 2!} \cdot (0.02)^2 \cdot (0.98)^8$$

$$= \frac{8! \times 9 \times 10}{8! \cdot 2!} (0.02)^2 \times (0.98)^8$$

$$= \frac{9 \times 10}{2} \times (0.02)^2 \times (0.98)^8 = \mathbf{0.0153} \quad \textbf{Ans.}$$

(*c*) Probability of exceedance *once* in *n years* is given by eq. (7.133), as:

$$p_{1,n} = n \cdot p \cdot q^{n-1}$$

when $n = 10$ years,

$$p_{1,10} = 10 \times 0.02 \times (0.98)^{10-1}$$

$$= 10 \times 0.02 \times (0.98)^9 = \mathbf{0.167} \quad \textbf{Ans.}$$

Example 7.55. *What return period you would adopt in the design of a culvert on a drain if you are allowed to accept only 5% risk of flooding in the 25 years of expected life of the culvert?*

Solution. 5% risk means that there is a probability of 0.05 for the design flood to occur *at least once* in successive 25 years. In other words, for 0.95 (*i.e.*95%) probability, the flood should not occur. Using

$$R = [1 - (1-p)^n], \text{ we have}$$

$$R = 0.05$$

$$p = \frac{1}{T} = ? \qquad \text{(where } T \text{ is the return period)}$$

$$n = 25 \text{ years}$$

$\therefore$ $0.05 = [1 - (1-p)^{25}]$,

or $(1-p)^{25} = 1 - 0.05 = 0.95$

or $1 - p = (0.95)^{\frac{1}{25}} = (0.95)^{0.01} = 0.99795$

or $p = 1 - 0.99795 = 0.00205$

$\therefore$ $T = \frac{1}{0.00205} = 487.8$, say 488 years. **Ans.**

7.50.3.1. Probability plotting on empirical relations. The main purpose of probability frequency analysis is to obtain a relation between the magnitude of a flood or a storm and its probability of exceedence. This analysis may be made by *empirical* or *analytical methods.*

The simplest *empirical technique* is : to arrange the given annual peak flood data in the descending order, and to assign the *ranking number (m)* to each flood. The severest flood will, thus, be placed at the top with its ranking as 1. The 2nd severest flood will be placed at number 2 position, and its ranking number will also be 2. The lightest flood peak will be placed at the last place (*N*th place), and its ranking will also be *N*.

The probability of an event equalling or exceeding is then computed by the *empirical* **California formula*** as:

$$p = \frac{m*}{N} \qquad ...(7.136)$$

and, the *recurrence interval* $(T) = \frac{1}{p} = \frac{N}{m}$ ...(7.137)

This relation $T = \frac{N}{m}$ was earlier used by us as eq. (7.18).

After the values of recurrence interval (T) for different floods are calculated, a graph can be plotted between frequencies and flood discharges; which can be extended if desired, as to extrapolate the value of flood magnitude corresponding to any high value of frequency. The flood discharge for any other frequency can also be determined from the plotted graph.

However, many a times, a graph is plotted between $\frac{100}{T}$ (called exceedence probability (p) in per cent or chance per cent) and the flood discharge. A simple graph paper can be used and the curve obtained is of the type, shown in Fig. 7.123. Sometimes,

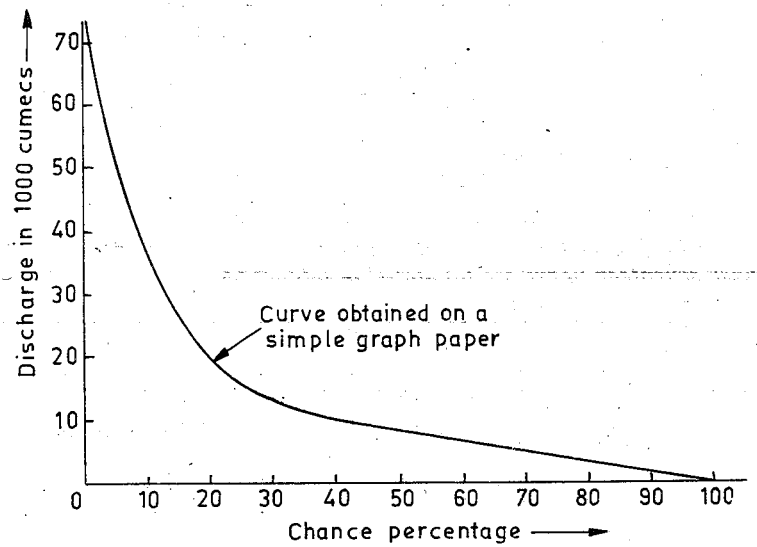


Fig. 7.123. Probability curve on a single graph paper.

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* Other formulas are :

(*i*) Welbull formula $p = \frac{m}{N+1}$ ...(7.138)

(*ii*) Hazen formula $p = \frac{m-0.5}{N}$ ...(7.139)

(*iii*) Chegodayev formula $p = \frac{m-0.3}{N+0.4}$ ...(7.140)

(*iv*) Blom formula $p = \frac{m-0.44}{N+0.12}$ ...(7.141)

logarithmic probability paper (such as shown in Fig. 7.124) may be used. The advantage of using such a paper lies in the fact, that especially throughout the lower percentages, this curve is much flatter (sometimes a straight line) than the one obtained on a simple graph paper. It can therefore, be extended more readily to cover the very low percentages.

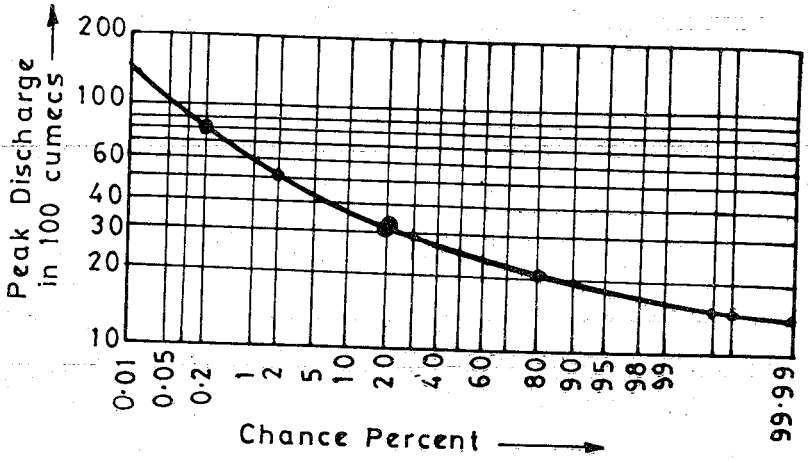


Fig.7.124. Probability curve on a logarithmic probability paper.

Sometimes, a semi log graph paper is used, and the frequency or return period (T) is plotted on the log scale on x-axis, and the flood discharge is plotted on the ordinary scale on y-axis, such as shown in Fig. 7.126. A straight line is generally obtained, and can be easily extended. This plotting is quite preferable as compared to the other two plottings.

Example 7.56. *Flood frequency records on a river have been collected for seventeen years starting from 1951 to 1967, and the peak values of the floods observed during each of these seventeen years are tabulated in table 7.66.*

Estimate the magnitude of flood having frequency equal to (a) 80 years (b) 400 years: by using an ordinary graph paper, and also by using a semi-log graph paper.

Table 7.66

Year	*Flood peak in cumecs*
1951	*3000*
1952	*4400*
1953	*6000*
1954	*3500*
1955	*2900*
1956	*4800*
1957	*3900*
1958	*3300*
1959	*6700*
1960	*5400*
1961	*4300*
1962	*3700*
1963	*4200*
1964	*9000*
1965	*4000*
1966	*3600*
1967	*5100*

Solution. The flood values are, first of all, arranged in their decreasing order in column (2) of table 7.67. and the table is completed by evaluating its remaining columns, *i.e.* (3), (4) and (5). Column (3) represents the number of times the year's flood is equalled or exceeded (*i.e. m*), and column (4) represents the frequency (T), and col. (5) represents the chance per cent of flood. The table is self-explanatory.

Table 7.67

Year	*Flood peaks in cumecs arranged in decreasing order*	*No. of times the year's peak is equalled or exceeded i.e. ranking of flood (m)*	*Frequency* $T = \frac{N}{m} = \frac{17}{Col.\ (3)}$	*Chance%* $= \frac{100}{T} = \frac{100}{Col.\ (4)}$
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1964	9000	1	17	5.9
1959	6700	2	8.5	11.8
1953	6000	3	5.67	17.7
1960	5400	4	4.25	23.5
1967	5100	5	3.4	29.4
1956	4800	6	2.83	35.3
1952	4400	7	2.43	41.2
1961	4300	8	2.12	47.1
1963	4200	9	1.89	52.9
1965	4000	10	1.7	58.9
1957	3900	11	1.55	64.7
1962	3700	12	1.42	70.5
1966	3600	13	1.31	76.5
1954	3500	14	1.21	82.4
1958	3300	15	1.13	88.8
1951	3000	16	1.06	94.1
1955	2900	17	1.0	100.0
	No. of floods $= N = 17$			

Case (*i*) *Using an ordinary graph paper.*

The values of chance per cent of column (5) are plotted against the values of flood peaks of column (1), on a simple graph paper, as shown in Fig. 7.125.

Case (a) The magnitude of a flood having a frequency of 80 years (or a chance per cent =100/80=1.25%); can be directly read from this graph (Fig. 7.125) as **11,000 cumecs. Ans.**

Case (b) Similarly, the magnitude of a flood having a frequency of 400 years (or a chance per cent $= \frac{1}{400} \times 100 = 0.25$) can be read from Fig. 7.125, **as 13,200 cumecs. Ans.**

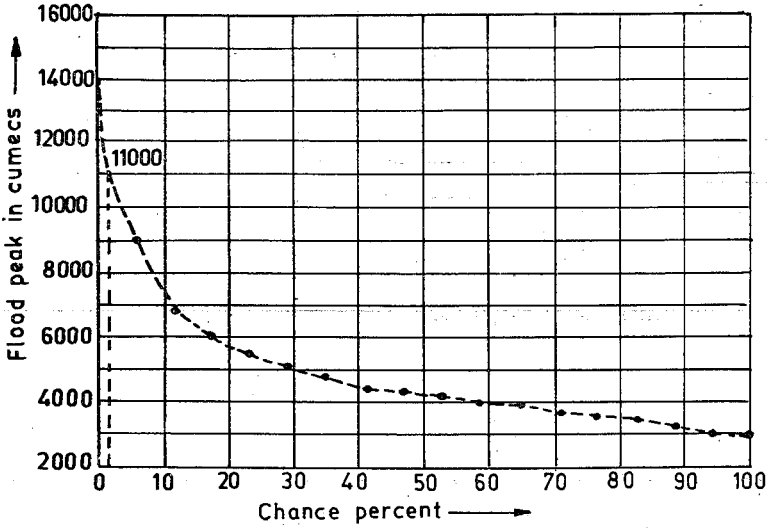


Fig. 7.125. Plot on an ordinary graph paper for example 7.56.

Case (ii) Using a Semi-log graph paper.

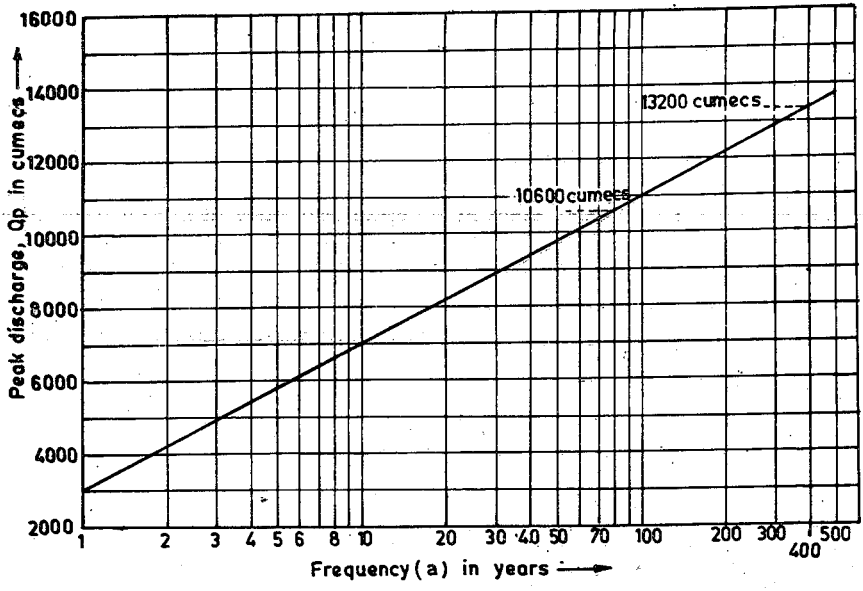


Fig. 7.126. Plot on semi-log paper for example 7.56.

The values of frequency or return period of col. (4) of Table 7.67, are plotted against the values of flood peaks of column (1), as shown in Fig. 7.126.

The magnitudes of floods having frequencies of 80 years and 400 years, are then read out from it, as *10,600 cumecs* and *13,200 cumecs*, respectively. **Ans.**

Example 7.57. *The flood frequency study on a river has shown that during 50 years, 150 floods have occurred. The magnitude of each flood was known, from which the number of floods falling between the two limits of flood peak were counted and tabulated in the table below :*

Flood peak limit in cumecs (1)	*No. of floods* (2)
650—709	*1*
710—779	*5*
780—844	*30*
845—914	*22*
915—989	*20*
990—1059	*18*
1060—1129	*14*
1130—1269	*10*
1270—1414	*13*
1415—1699	*8*
1700—1979	*3*
1980—2499	*5*
2500—2900	*1*
	$\Sigma = 150$

Evaluate the magnitude of flood having a frequency of

(a) 15 years,

(b) 30 years; and

(c) 80 years

use the simple probability method.

Solution. Let us, first of all, extend the given table by evaluating columns (3) and (4) as described below, and as shown in Table 7.68.

In column (3), the number of times the given flood is equalled or exceeded is written; and in column (4), chance percentages are calculated by dividing column (3) values by the total number of floods, (*i.e.* 150) and multiplying by 100. The table is otherwise self-explanatory.

Table 7.68

Flood peak limit in cumecs (1)	*No. of floods* (2)	*No. of times the flood represented by the lower value of col. (1) is equalled or exceeded i.e. the ranking of the flood (m)* (3)	*Chance Percentage* $= \frac{100}{T} = \frac{100\,m}{N} = \frac{Col.\,(3) \times 100}{Total\ no.\ of\ floods} = \frac{Col.\,(3)}{150} \times 100$ (4)
650—709	1	150	100.00
710—779	5	149	99.33
780—844	30	144	96.00
845—914	22	114	76.00
915—989	20	92	61.33
990—1059	18	72	48.00
1060—1129	14	54	36.00
1130—1269	10	40	26.67
1270—1414	13	30	20.00
1415—1699	8	17	11.33
1700—1979	3	9	6.00
1980—2499	5	6	4.00
2500—2900	1	1	0.67
	$\Sigma = N = 150$		

The chance percentages calculated in column (4) are plotted on a graph paper against the lower flood peak values of column (1) as shown in Fig. 7.127.

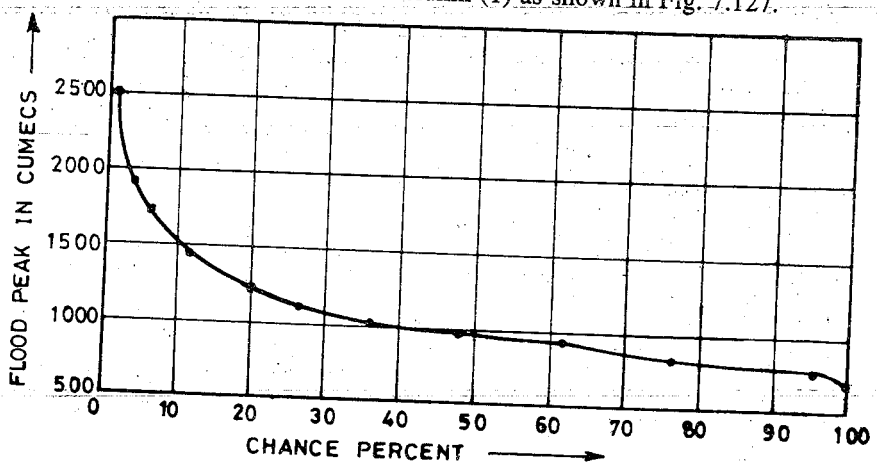


Fig. 7.127. Plot on simple graph paper for example 7.57.

This graph will be used for evaluating the magnitude of a flood having a given frequency.

Case. (*a*) When frequency = 15 years

In 50 years, No. of floods occurred = 150

$\therefore$ In 15 years, No. of floods likely to occur $= \frac{150}{50} \times 15 = 45.$

The greatest flood expected in this period of 15 years, will have a chance per cent.

$$= \frac{\text{Ranking of this flood}}{\text{Total No. of floods}} \times 100 = \frac{1}{45} \times 100 = 2.2\%$$

The magnitude of a flood corresponding to a chance per cent of 2.2, can be read out from the graph, Fig. 7.127, as **2225 cumecs. Ans.**

Case (*b*) When frequency = 30 years

In 50 years, No. of floods occurred = 150

$\therefore$ In 30 years, No. of floods likely to occur $= \frac{150}{50} \times 30 = 90$

The greatest flood expected in this period of 30 years, will be having its ranking as 1, and a chance per cent $= \frac{1}{90} \times 100 = 1.11\%$

Corresponding to this value of chance per cent, *i.e.* 1.11, the flood peak can be read out from Fig. 7.127 as **2350 cumecs. Ans.**

Case (*c*) When frequency = 80 years.

In 50 years, No. of floods occurred = 150

$\therefore$ In 80 years, No. of floods likely to occur $= \frac{150}{50} \times 80 = 240$

Chance percentage of the greatest flood occurring in this period of 80 years

$$= \frac{1}{240} \times 100 = 0.416.$$

Corresponding to this chance per cent, of 0.416, the flood peak can be read out from Fig. 7.127, as approximately **2600 cumecs. Ans.**

7.50.3.2. Theoretical probability distribution methods. The simple empirical procedure explained above can give good results for small extra-polations only; and as the extra-polation increases, the error also increases. However, for obtaining better results for larger extra-polations, theoretical probability distributions have to be used.

Thus, for determining extreme flood events, specific extreme value distributions are assumed, and the required statistical parameters are determined from the available data, from which the flood magnitude (Q) for any specific return period (T) can be determined.

The basic statistical equation, is given by

$$X = \bar{X} + K \cdot \sigma \qquad \ldots(7.142)$$

where X is any variable*

$\bar{X}$ = Mean of variate

σ = Standard deviation of the variate

K = A constant.

This equation has been shown by Chow to be the governing equation for almost all the theoretical frequency distribution functions.

* Such as discharge.

The Gumbel's extreme value function or log Pearson's Type III distribution, can thus, be well expressed by the above equation. Gumbel's distribution is largely used to extrapolate flood data, and is discussed below:

Gumbel's Method. *According to Gumbel's theory of extreme events, the probability of occurrence of an event equal to or larger than a value X_0 is given by :*

$$p\,(X \geq X_0) = 1 - e^{-e^{-y}} \qquad ...(7.143)$$

where y = another dimensional variable given by

$$y = \alpha\,(X - \beta) \qquad ...(i)$$

where $\alpha = \dfrac{1.2825}{\sigma}$ ...(ii)

$$\beta = \overline{X} - 0.45005\,\sigma \qquad ...(iii)$$

σ = Standard deviation of variate X.

$$\therefore \quad y = \frac{1.2825}{\sigma}\left[X - \overline{X} + 0.45005\,\sigma\right]$$

or
$$y = \frac{1.2825}{\sigma}\left[X - \overline{X}\right] + 0.577 \qquad ...(7.144)$$

In actual practice, it is the value of $\overline{X}$ for a given probability (p) that is required; and as such, equation (7.143) is transposed as

$y_{(p)}$ = value of variate y for a given probability (p)

$$= -\ln\left[-\ln\,(1-p)\right] \qquad ...(7.145)$$

Using $p = \dfrac{1}{T}$, we have

$$\boxed{y_{(T)} = -\left[\ln \cdot \ln \frac{T}{T-1}\right]} \qquad ...(7.146)$$

Also from eq. (7.144), we have

$$y_{(T)} = \frac{1.2825}{\sigma}\left[X_{(T)} - \overline{X}\right] + 0.577 \qquad ...(7.144\,(a))$$

where $X_{(T)}$ = Value of X for a return period of T.

or
$$y_{(T)} - 0.577 \frac{\sigma}{1.2825} = X_{(T)} - \overline{X}$$

or
$$X_{(T)} = \overline{X} + \left(\frac{y_{(T)} - 0.577}{1.2825}\right)\sigma$$

or
$$\boxed{X_{(T)} = \overline{X} + K \cdot \sigma} \qquad ...(7.147)$$

where
$$K = \frac{y_{(T)} - 0.577}{1.2825} \text{ for } N \to \infty \qquad ...(7.148)$$

where $y_{(T)}$ is given by Eq. (7.146); and

K is known as **frequency factor**

Note I. Eq. (7.147) above is of the same form as the general basic statistical Eqn. (7.142) is

Note II. Eqns. (7.146), (7.147), and (7.148) constitute the basic Gumbel's equations, and are applicable to a sample of infinite size, *i.e.* $N \to \infty$.

When N is smaller, and is of a finite value, then eq. (7.148) is modified to vary K, as:

$$K = \frac{y_{(T)} - \bar{y}_n}{s_n} \qquad \text{...[7.148(a)]}$$

where $\bar{y}_n$ = reduced mean, depending on N, values of which are given in Table 7.69, maximum value is 0.577 at $N \to \infty$

s_n = reduced standard deviation, depending on N, values of which are given in Table 7.70. Maximum value = 1.2825 at $N \to \infty$.

Table 7.69. Reduced Mean $\bar{y}_n$ in Gumbel's Extreme Value Distribution

N = Sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.4952	0.4996	0.5035	0.5070	0.5100	0.5128	0.5157	0.5181	0.5202	0.5520
20	0.5236	0.5252	0.5268	0.5283	0.5296	0.5309	0.5320	0.5332	0.5343	0.5353
30	0.5362	0.5371	0.5380	0.5388	0.5396	0.5402	0.5410	0.5418	0.5424	0.5430
40	0.5436	0.5442	0.5448	0.5453	0.5458	0.5463	0.5468	0.5473	0.5477	0.5481
50	0.5485	0.5489	0.5493	0.5497	0.5501	0.5504	0.5508	0.5511	0.5515	0.5518
60	0.5521	0.5524	0.5527	0.5530	0.5533	0.5535	0.5538	0.5540	0.5543	0.5545
70	0.5548	0.5550	0.5552	0.5555	0.5557	0.5559	0.5561	0.5563	0.5565	0.5567
80	0.5569	0.5570	0.5572	0.5574	0.5576	0.5578	0.5580	0.5581	0.5583	0.5585
90	0.5586	0.5587	0.5589	0.5591	0.5592	0.5593	0.5595	0.5596	0.5598	0.5599
100	0.5600									

$y_n = 0.5770$ at $N = \infty$

Table 7.70. Reduced Standard Deviation s_n in Gumbel's Extreme Value Distribution

N = Sample size

N	0	1	2	3	4	5	6	7	8	9
10	0.9496	0.9676	0.9833	0.9971	1.0095	1.0206	1.0316	1.0411	1.0493	1.0565
20	1.0628	1.0696	1.0754	1.0811	1.0864	1.0915	1.0961	1.1004	1.1047	1.1086
30	1.1124	1.1159	1.1193	1.1226	1.1255	1.1285	1.1313	1.1339	1.1363	1.1388
40	1.1413	1.1436	1.1458	1.1480	1.1499	1.1519	1.1538	1.1557	1.1574	1.1590
50	1.1607	1.1623	1.1638	1.1658	1.1667	1.1681	1.1696	1.1708	1.1721	1.1734
60	1.1747	1.1759	1.1770	1.1782	1.1793	1.1803	1.1814	1.1824	1.1834	1.1844
70	1.1854	1.1863	1.1873	1.1881	1.1890	1.1898	1.1905	1.1915	1.1923	1.1930
80	1.1938	1.1945	1.1953	1.1959	1.1967	1.1973	1.1980	1.1987	1.1994	1.2001
90	1.2007	1.2013	1.2020	1.2026	1.2032	1.2038	1.2044	1.2049	1.2055	1.2060
100	1.2065									

$s_n = 1.2825$ at $N = \infty$

Example 7.58. *For a river, the estimated flood peaks for two return periods by the use of Gumbel's method, are given below*

Return period (years)	*Peak flood (m^3/s)*
100	*485*
50	*445*

What flood discharge in this river will have a return period of 1000 years.

(U.P.S.C.; Civil Services, 1990)

Solution. Using Gumbel's Eqns., we have eqn. (7.147) as:

$$X_{(T)} = \bar{X} + K\sigma$$

where K is given by general eqn. (7.148 *a*) as :

$$K = \frac{y_{(T)} - \bar{y}_n}{s_n}$$

$\bar{y}_n$ and s_n remain the same for one analysis, because N is fixed in one analysis.

$$\therefore \qquad X_{(100)} = \bar{X} + \left[\frac{y_{(100)}}{s_n} - \frac{\bar{y}_n}{s_n}\right]\sigma = 485 \text{ m}^3/\text{s (given)} \qquad ...(i)$$

and $$X_{(50)} = \bar{X} + \left[\frac{y_{(50)}}{s_n} - \frac{\bar{y}_n}{s_n}\right]\sigma = 445 \text{ m}^3/\text{s (given)} \quad ...(ii)$$

Substracting (*ii*) from (*i*), we get

$$\left[\frac{y_{(100)}}{s_n} - \frac{y_{(50)}}{s_n}\right]\sigma = 40 \text{ m}^3/\text{s} \quad ...(iii)$$

$y_{(T)}$ is given by eq. (7.146) as :

$$y_{(T)} = -\left[l_n\, l_n \cdot \frac{T}{T-1}\right]$$

$$\therefore \quad y_{(100)} = -\left[l_n\, l_n \frac{100}{99}\right] = 4.60015$$

and $$y_{(50)} = -\left[l_n\, l_n \frac{50}{49}\right] = 3.90194$$

$\therefore$ Substituting in (*iii*), we get

$$(4.60015 - 3.90194) \cdot \frac{\sigma}{s_n} = 40$$

or $$\frac{\sigma}{s_n} = 57.2894$$

Also, for given 1000 years T, we have

$$y_{(1000)} = -\left[l_n\, l_n \frac{1000}{999}\right] = 6.90726$$

Also, from the basic eqns. For 1000 years and 100 years, we have,

$$\left[y_{(1000)} - y_{(100)}\right]\frac{\sigma}{s_n} = X_{(1000)} - X_{(100)}$$

Substituting values, we get

$$[6.90726 - 4.60015]\, 57.2894 = X_{(1000)} - 485$$

or $$132.17 = X_{(1000)} - 485$$

or $$X_{(1000)} = \mathbf{617.17\ m^3/s} \quad \textbf{Ans.}$$

Example 7.59. *Solve example 7.56 by using Gumbel's probability method by using analytical technique.*

Solution. The given table 7.66 is extended in Table 7.71, which is otherwise self-explanatory.

Table 7.71

Year	*Flood peaks in cumecs arranged in decreasing order (X)*	*No. of times the year's peak is equalled or exceeded, i.e. ranking of flood (m)*	*Recurrence interval, i.e. frequency* $T=\frac{N+1^*}{m}=\frac{17+1}{m}=\frac{18}{m}=\frac{18}{Col.\ (3)}$	*Chance per cent or exceedence probability* $p=\frac{100}{T}=\frac{100}{Col.\ (4)}$	X^2
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
1964	9,000	1	18.0	5.55	81.0×10^6
1959	6,700	2	9.0	11.1	44.8×10^6
1953	6,000	3	6.0	16.67	36.0×10^6
1960	5,400	4	4.5	22.2	29.2×10^6
1967	5,100	5	3.6	27.78	25.9×10^6
1956	4,800	6	3.0	33.33	23.1×10^6
1952	4,400	7	2.56	3.83	19.4×10^6
1961	4,300	8	2.25	44.44	18.5×10^6
1963	4,200	9	2.0	50.0	17.6×10^6
1965	4,000	10	1.8	55.5	16.0×10^6
1957	3,900	11	1.64	61.11	15.2×10^6
1962	3,700	12	1.5	66.67	13.7×10^6
1966	3,600	13	1.4	72.22	13.0×10^6
1954	3,500	14	1.3	77.77	12.2×10^6
1958	3,300	15	1.2	83.33	10.9×10^6
1951	3,000	16	1.13	83.88	9.0×10^6
1955	2,900	17	1.06	94.44	8.4×10^6
$N=17$	$\Sigma X=77,800$				$\Sigma X^2=393.9\times10^6$

The desired analytical solution is carried out as given below:

Mean peak discharge $\bar{X}=\frac{\Sigma X}{N}=\frac{77,800}{17}=4,580=4.58\times10^3$

Mean of squares $=\bar{X}^2=\frac{\Sigma X^2}{n}=\frac{393.9\times10^6}{17}=23.2\times10^6$

Standard deviation, $\sigma=\sqrt{\frac{N}{N-1}\left[\bar{X}^2-(\bar{X})^2\right]}$

$$=\sqrt{\frac{17}{17-1}\left[23.2\times10^6-(4.58\times10^3)^2\right]}$$

* Weibull Probability equation $p=\frac{m}{N+1}$; *i.e.* $T=\frac{N+1}{m}$ is used in Gumbel's method.

$$= \sqrt{\frac{17\times10^6}{16}(23.2-20.9)}$$

$$= \sqrt{\frac{17\times10^6}{16}(2.3)} = \sqrt{2.44\times10^6}$$

$$= \mathbf{1.56\times10^3}$$

Now, $K = \dfrac{y_{(T)} - \bar{y}_n}{s_n}$

For T = 80 years;

$$y_{(T)} = -\left[\ln\ln\cdot\frac{T}{T-1}\right]$$

$$\therefore \quad y_{(80)} = -\left[\ln\ln\frac{80}{79}\right] = 4.37574$$

$\bar{y}_n$ for 17 years sample is found from table 7.69 as equal to 0.5181

s_n for 17 years sample is found from table 7.70= 1.0411

$$\therefore \quad K_{(80)} = \frac{4.37574 - 0.5181}{1.0411} = \mathbf{3.705}$$

Similarly, for T = 400 years, we have

$$y_{(400)} = -\left[\ln\ln\cdot\frac{400}{399}\right] = 5.99021$$

$$\bar{y}_n = 0.5181$$

$$s_n = 1.0411$$

$$\therefore \quad K_{(400)} = \frac{5.99021 - 0.5181}{1.0411} = \mathbf{5.256}$$

The expected peak discharge for a flood having a frequency of 80 years is then given by:

$$X = \bar{X} + K\cdot\sigma$$

$$\therefore \quad X_{(80)} = 4.58\times10^3 + 3.705\times1.56\times10^3$$

$$= (4.58 + 5.78)\ 10^3$$

$$= 10.36\times10^3 = \mathbf{10{,}360\ cumecs.}\quad \mathbf{Ans.}$$

The expected peak discharge for a flood having a frequency of 400 years, is similarly given by:

$$X_{(400)} = 4.58\times10^3 + 5.256\times1.56\times10^3$$

$$= (4.58 + 8.20)\ 10^3$$

$$= 12.78\times10^3 = \mathbf{12{,}780\ cumecs.}\quad \mathbf{Ans.}$$

Note. In order to avoid large scale calculations involved in calculating K values in Gumbel's equation, as carried out in examples 7.57 and 7.58, readymade table has been drawn, which reflects the values of K for a given sample size (N) and frequency (T). Such values are given in table 7.72, and their use is reflected in example 7.60.

Table 7.72. Frequency Factor (K) for Gumbel's Method

Sample size (N) in years	Frequency (T) in years										
	5	10	15	20	25	30	50	60	75	100	1000
15	0.967	1.703	2.117	2.410	2.632	2.823	3.321	3.501	3.721	4.005	6.265
20	0.919	1.625	2.023	2.302	2.517	2.690	3.179	3.352	3.563	3.836	6.006
25	0.888	1.575	1.963	2.235	2.444	2.614	3.088	3.257	3.463	3.729	5.848
30	0.866	1.541	1.922	2.188	2.393	2.560	3.026	3.191	3.393	3.653	5.727
35	0.851	1.516	1.891	2.152	2.354	2.520	2.979	3.142	3.341	3.598,	
40	0.838	1.495	1.866	2.126	2.326	2.489	2.943	3.104	3.301	3.554	5.576
45	0.829	1.478	1.847	2.104	2.303	2.464	2.913	3.078	3.268	3.520	
50	0.820	1.466	1.831	2.086	2.283	2.443	2.889	3.048	3.241	3.491	5.478
55	0.813	1.455	1.818	2.071	2.267	2.426	2.869	3.027	3.219	3.467	
60	0.807	1.446	1.806	2.059	2.253	2.411	2.852	3.008	3.200	3.446	
65	0.801	1.437	1.796	2.048	2.243	2.398	2.837	2.992	3.183	3.429	
70	0.797	1.430	1.788	2.038	2.230	2.387	2.824	2.979	3.169	3.413	5.359
75	0.792	1.423	1.780	2.029	2.220	2.377	2.812	2.967	3.155	3.400	
80	0.788	1.417	1.773	2.020	2.212	2.368	2.802	2.956	3.145	3.387	
85	0.785	1.413	1.767	2.013	2.205	2.361	2.793	2.946	3.135	3.376	
90	0.782	1.409	1.762	2.007	2.198	2.353	2.785	2.938	3.125	3.367	
95	0.780	1.405	1.757	2.002	2.193	2.347	2.777	2.930	3.116	3.357	
100	0.779	1.401	1.752	1.993	2.187	2.341	2.770	2.922	3.109	3.349	2.61

Example 7.60. *The following data for the yearly peak floods have been observed and kept at Okhla Weir in New Delhi for 70 years from 1920 to 1989 (when the weir was replaced by a new barrage at a downstream point). Determine from Gumbel's analytical as well as graphical methods, the expected peak flows for floods having frequencies of (i) 50 years, (ii) 75 years, (iii) 100 years. Also determine the estimated frequency of a flood having a peak value equal to 3 lakh cusecs.*

Date (1)	Peak discharge in cusecs (2)
1.8.1920	58,743
16.8.1921	58,369
13.9.1922	67,953
4.10.1923	49,594
2.10.1924	2,05,183
6.8.1925	87,505
17.8.1926	60,302
3.8.1927	78,102
25.8.1928	35,885
21.8.1929	77,361
13.8.1930	64,406
1.10.1931	48,762
13.9.1932	88,775
22.9.1933	1,05,408
29.9.1934	78,104
21.8.1935	49,069
4.9.1936	46,907
21.9.1937	71,061
14.8.1938	96,096
29.7.1939	30,313
22.8.1940	38,230

...Contd.

Date (1)	Peak discharge in cusecs (2)
21.8.1941	43,200
12.8.1942	90,329
2.9.1943	86,029
4.9.1944	35,365
30.9.1945	70,789
28.7.1946	64,212
30.9.1947	1,33,672
23.8.1948	77,775
5.8.1949	56,125
7.9.1950	68,510
27.8.1951	59,209
26.8.1952	81,421
16.8.1953	68,892
29.9.1954	71,914
9.10.1955	1,30,324
16.10.1956	1,77,631
18.9.1957	90,718
7.8.1958	93,254
13.9.1959	88,022
27.8.1960	80,386
19.8.1961	1,00,985
26.9.1962	1,32,245
19.9.1963	1,17,991
29.9.1964	1,26,958
20.7.1965	61,980
18.8.1966	1,14,852
30.8.1967	1,55,073
24.7.1968	98,084
9.8.1969	71,058
16.8.1970	67,752
10.8.1971	1,32,000
18.9.1972	88,127
18.7.1973	85,327
7.8.1974	78,564
12.9.1975	1,03,714
22.8.1976	1,46,569
7.8.1977	1,05,849
6.9.1978	2,19,302
24.7.1979	77,048
4.8.1980	1,18,264
5.8.1981	94,808
12.8.1982	65,028
30.8.1983	1,17,450
7.9.1984	43,326
13.10.1985	94,621
15.8.1986	78,906
28.8.1987	28,275
27.9.1988	2,33,918
30.8.1989	1,61,692

Solution. The given flood peaks are arranged in descending order in column (2) of Table 7.73. The ranking of floods (m) are marked in col. (3). The frequencies (T) are then calculated in col. (4) using $T = \frac{N+1}{m}$. The chance percentages $= \frac{100}{T}$ are then

calculated in col. (5), and finally Q_p^2 are calculated in col. (6). The table is otherwise self-explanatory.

Table 7.73

Year and date	*Peak discharge in 10^5 cusecs arranged in descending order Q_p*	*Ranking of flood or order number (m)*	*Frequency or return period* $T = \frac{N+1}{m} = \frac{70+1}{m}$ $\frac{58}{\text{Col. (3)}}$	*Chance Percentage or exceedence Probability* $= \frac{100}{T} = \frac{100}{\text{Col (4)}}$	$Q_p^2 \times 10^{-10}$
(1)	(2)	(3)	(4)	(5)	(6)
27.9.88	2.33	1	71	1.41	5.43
6.9.78	2.19	2	35.5	2.82	4.80
2.10.24	2.05	3	23.67	4.22	4.20
16.10.56	1.78	4	17.75	5.63	3.17
30.8.89	1.62	5	14.20	7.04	2.62
30.8.67	1.55	6	11.83	8.45	2.40
22.8.76	1.47	7	10.14	9.86	2.16
30.9.47	1.34	8	8.88	11.27	1.79
10.8.71	1.33	9	7.89	12.67	1.77
26.9.62	1.32	10	7.10	14.08	1.74
9.10.55	1.30	11	6.45	15.50	1.69
21.9.64	1.27	12	5.92	16.89	1.61
4.8.80	1.18	13	5.46	18.31	1.39
14.9.63	1.18	14	5.07	19.72	1.39
30.8.83	1.17	15	4.73	21.14	1.37
18.8.66	1.15	16	4.44	22.52	1.32
7.8.77	1.06	17	4.18	23.92	1.12
22.9.33	1.05	18	3.94	25.38	1.10
12.9.75	1.04	19	3.74	26.74	1.08
19.8.61	1.01	20	3.55	28.17	1.02
24.7.68	0.98	21	3.38	29.59	0.96
5.8.81	0.95	22	3.23	30.96	0.90
13.10.85	0.95	23	3.09	32.36	0.90
7.8.58	0.93	24	2.96	33.78	0.86
18.9.57	0.91	25	2.84	35.21	0.83
12.8.42	0.90	26	2.77	36.10	0.81
13.9.32	0.89	27	2.63	38.02	0.79
18.9.72	0.88	28	2.54	39.37	0.77
13.9.59	0.88	29	2.45	40.82	0.77
6.3.25	0.88	30	2.37	42.19	0.77
14.8.38	0.86	31	2.29	43.67	0.74
2.9.43	0.86	32	2.22	45.04	0.74
8.3.73	0.85	33	2.15	46.51	0.72
8.5.52	0.81	34	2.09	47.85	0.66
27.8.60	0.80	35	2.03	49.26	0.64

Contd....

(1)	(2)	(3)	(4)	(5)	(6)
15.8.86	0.79	36	1.97	50.76	0.62
7.8.74	0.79	37	1.92	52.08	0.62
29.3.34	0.78	38	1.87	53.48	0.61
3.8.27	0.78	39	1.82	54.95	0.61
23.8.48	0.78	40	1.78	56.18	0.61
21.8.29	0.77	41	1.73	57.80	0.59
24.7.79	0.77	42	1.69	59.17	0.59
29.9.54	0.72	43	1.65	60.61	0.52
9.8.69	0.72	44	1.61	62.11	0.52
21.9.37	0.71	45	1.58	63.29	0.50
30.9.45	0.71	46	1.54	64.93	0.50
15.8.53	0.69	47	1.51	66.23	0.48
7.9.50	0.69	48	1.48	67.57	0.48
13.9.22	0.68	49	1.45	68.97	0.46
16.8.70	0.68	50	1.42	70.42	0.46
13.8.82	0.65	51	1.39	71.94	0.42
13.8.30	0.64	52	1.37	72.99	0.41
28.7.46	0.64	53	1.34	74.63	0.41
20.7.65	0.62	54	1.31	76.34	0.38
17.8.26	0.60	55	1.29	77.52	0.36
27.8.51	0.59	56	1.27	78.74	0.35
1.8.20	0.59	57	1.25	80.00	0.35
16.8.21	0.58	58	1.22	81.97	0.34
5.8.49	0.56	59	1.20	83.33	0.31
4.10.23	0.50	60	1.18	84.75	0.25
21.8.35	0.49	61	1.16	86.21	0.24
1.10.31	0.49	62	1.15	86.96	0.24
21.4.46	0.47	63	1.13	88.50	0.22
7.9.84	0.43	64	1.11	90.09	0.18
21.8.41	0.43	65	1.09	91.74	0.18
22.8.40	0.38	66	1.08	92.59	0.15
25.8.20	0.36	67	1.06	94.34	0.13
4.9.44	0.35	68	1.04	96.15	0.12
29.7.39	0.30	69	1.03	97.09	0.09
28.8.87	0.28	70	1.01	99.00	0.08
Σ	$\Sigma Q_p =$ 62.73×10^5				$\Sigma Q_p^2 =$ 68.41×10^{10}

Using Gumbel's Analytical Method :

Mean peak discharge

$$= \overline{Q}_p = \frac{\Sigma Q_p}{N} = \frac{62.73 \times 10^5}{70} = 0.896 \times 10^5.$$

Mean of squares $= \overline{Q}_p^2 = \frac{\Sigma Q_p^2}{N} = \frac{68.41 \times 10^{10}}{70} = 0.977 \times 10^{10}$

Standard deviation,

$$\sigma = \sqrt{\frac{N}{N-1}\left[\overline{Q_p^2} - (\overline{Q}_p)^2\right]}$$

$$= \sqrt{\frac{70}{70-1}\left[0.977 \times 10^{10} - (0.896 \times 10^5)^2\right]}$$

$$= \sqrt{\frac{70}{69}\left[0.977 - 0.803\right] \times 10^{10}}$$

$$= \sqrt{\frac{70}{69} \times 0.174 \times 10^{10}} = \mathbf{0.42 \times 10^5}$$

Gumbel's analytical equation for finding discharge is

$$Q_p = \overline{Q}_p + K \cdot \sigma$$

The values of frequency factor (K) for different frequencies and a sample size of 70 years are read out from Table 7.72 as

Frequency	*Value of K*
50 years	2.824
75 years	3.169
100 years	3.413

These values of K are used for calculating discharges at various frequencies, as below:

Peak discharge for 50 years frequency

$$= (Q_p)_{50} = 0.896 \times 10^5 + 2.824 \times 0.42 \times 10^5$$

$$= \mathbf{2.08 \times 10^6 \text{ cusecs.}} \quad \textbf{Ans.}$$

Peak discharge for 75 years frequency

$$= (Q_p)_{75} = 0.896 \times 10^5 + 3.169 \times 0.42 \times 10^5$$

$$= \mathbf{2.23 \times 10^5 \text{ cusecs.}} \quad \textbf{Ans.}$$

Peak discharge for 100 years frequency

$$= (Q_p)_{100} = 0.896 \times 10^5 + 3.413 \times 0.42 \times 10^5$$

$$= \mathbf{2.33 \times 10^5 \text{ cusecs.}} \quad \textbf{Ans.}$$

Frequency of 3 lakh cusecs discharge is calculated as below. K for 3 lakh cusecs discharge is given by

$$3 \times 10^5 = 0.896 \times 10^5 + K \times 0.42 \times 10^5$$

or $$K = \frac{3 - 0.896}{0.42} = 5.01.$$

From Table 7.72, the frequency corresponding to a value of $K = 5.01$ and a sample size of 70 years is calculated by straight line interpolation as = **838 years. Ans.**

Graphical Method. A graph is plotted and extended between frequency (T) and peak discharge (Q_p) on a semi-log paper, as shown in Fig. 7.128. The expected peak flows for different frequencies, such as 50, 75 and 100 years are read out from this graph as 2.19×10^5, 2.36×10^5, 2.50×10^5 cusecs, respectively. The frequency of 3 lakh cusecs flood is found to be 320 years as against the value of 838 years computed by Gumbel's extrapolation, which shows the unreliablity contained in the computed frequency of

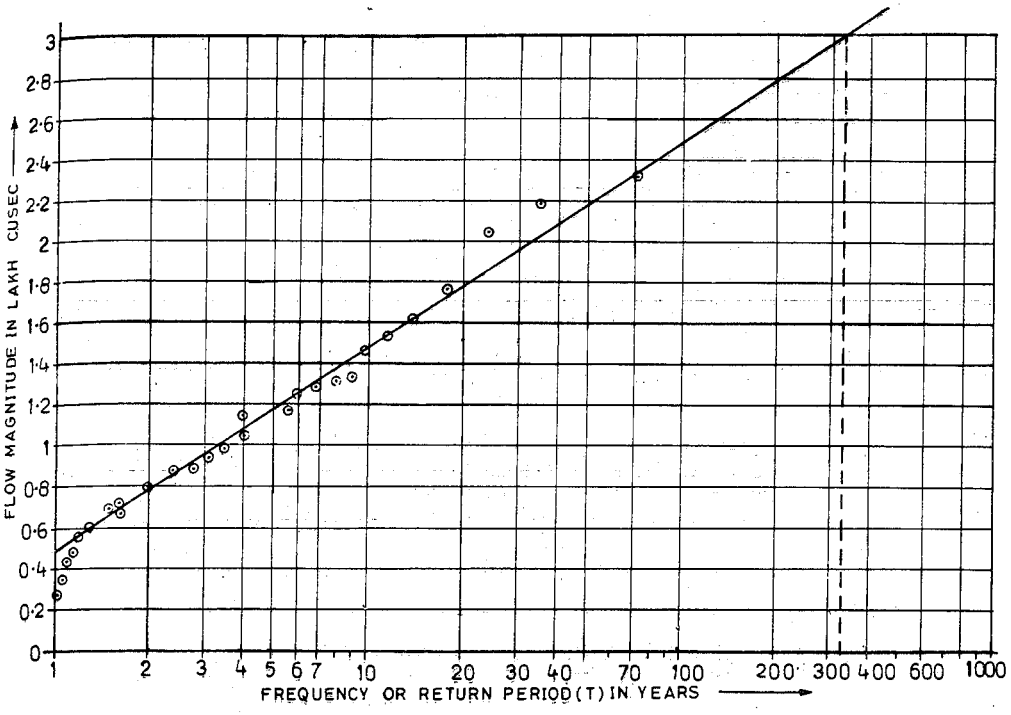


Fig. 7.128. Plotting for example 7.60.

extrapolated higher flood value of 3 lakh cusecs. The values obtained from analytical as well as graphical methods are tabulated below.

Frequency in years (1)	*Expected peak flow in cusec (from analytical) solution* (2)	*Expected peak flow in cusec (from graph)* (3)
50	2.19×10^5	2.08×10^5
75	2.36×10^5	2.23×10^6
100	2.50×10^5	2.33×10^6

7.50.3.3. Annual series and Partial series. The flood frequency analysis carried out in the last pages was based on considering only the peak flood for each year, for a large number of consecutive years. Such annual peak flood data*, when used to compute anticipated flood of a given frequency (recurrence interval), constitute what is known as **annual series.** *Thus, in an annual series, the largest flood observed in each year is only considered, and the second or lower down events of each year, which even if greater than the annual peak of the other included years, are ignored.*

Sometimes however, all the major events, irrespective of their being occurring a number of times in one single year, are considered. In compiling such a hydrological data, therefore, all the floods above a particular selected **base value,** are included Hence

* This also applies to rainfall data, when analysing rainfalls.

in such a series, even two or three floods* occurring in one single year may get included. However, the base value is so chosen, as not to include more than two or three floods of a single year. Such a compiled hydrological data is called a **partial series** or **partial duration series.**

While including the events in such a partial series, care should be taken to see that only the independent events get included. Hence, if one flood is included, then the second flood which has occured in the vicinity of the first flood and included by the first, should not be included. *Thus, out of two consecutive events, which are inter dependent, only the higher one should be included.*

Since floods are generally interdependent, partial series are rarely used for analysing flood data. However, since rainfalls generally occur quite independently, such series are mostly adopted for analysis of rainfall data.

It may also be clarified that the annual series analysis is generally adopted for determining events of larger recurrence intervals, say more than 20 years or so, as in the estimation of extreme floods for the design of hydraulic structures, such as weirs, barrages, dams, levees, etc. On the other hand, partial series analysis is generally adopted in estimating events of frequent occurrence, having say return period of less than 5 years or so, as may be the case in the design of *urban drains, coffer dams for river diversion, etc.*

In fact, the recurrence interval of an event obtained by annual series (T) and that obtained by partial duration series (T') have been related by the equation.

$$T' = \frac{1}{\log_n T - \log_n (T-1)} \qquad ...(7.149)$$

From this equation, it can be inferred that the difference between T and T' is significant only for $T < 10$ years; and this difference is negligibly small for $T > 20$ years. Hence, partial series does not serve any special use for estimating events, having frequency of more than 20 years or so.

7.50.3.4. Limitations of the probability methods. Forecasting the future flood by any of the probability methods involves the use of the past records. Greater the number of available past records, greater will be the accuracy. For example, if we want to predict the magnitude of a flood having a frequency of 20 years, and only 20 years of records are available, then the error is likely to be great, because our result is based on one sample alone. If 200 years of records would have been available for the same determination, then our result would have been based on 10 samples, and hence, must have been more precise. If 2,000 years of records would have been available, then our result would have been based on 100 samples, and hence would have been very precise, with almost no possibility of an error.

Under the same argument, we can easily state that if we want to determine a flood having a frequency of 100 years from 50 years of records, then the result obtained will be very doubtful, because even one full sample is not available. Normally, 10 samples are sufficient to give reasonable results. The results will be much more doubtful, if we want to determine a flood having a frequency of 1000 years from a record of 50 years.

Generally, too long records are not available and, therefore, it becomes *evident that high frequency flood determination cannot be made precisely by the use of Probability*

* This also applies to rainfall data, when analysing rainfalls.

methods. Low frequency flood determinations can no doubt be made easily and correctly by this method.

7.50.4. Determination of Flood Hydrograph By Unit Hydrograph Method. The unit hydrograph method, as stated earlier, is a very useful and a reliable method for computing design flood for a project, provided the basin is small and medium in size, say **up to 5000 sq. km.** *For larger basins,* the basin will however, have to be subdivided into smaller sub-basins (stream wise) and to compute different unit hydrographs and flood hydrographs for all these sub basins. All the flood hydrographs of the sub basins will then have to be routed up to the project site by *river routing techniques**. If reservoirs also happen to exist on the travel path, then the flood hydrographs will also have to be routed through them, by using *reservoir routing techniques* (discussed in Chapter 18). The cumulative impact of all the design flood hydrographs will finally be computed at the project site to determine the design flood for such large basins. Such long calculations make this method unattractive and unprecise for large basins, where probability methods, particularly the Gumbel's method, are preferably used to estimate the peak flood discharge for a specified frequency. However, full flood hydrograph cannot be obtained by these probability methods, as is given by the U.H. method, and as may be required in some projects where total volume of water becomes of paramount importance.

In order to estimate the design flood hydrograph by the U.H. method, we have to first of all evaluate the effective *design storm* (*i.e.* hyetograph of excess rain), which can be obtained by subtracting the initial loss and infiltration from the design storm. The **design storm** may be *(i) the storm of any selected design frequency*; or *(ii) the probable maximum storm*; or *(iii) the standard project storm* (usually considered to be equal to the *maximum observed storm of the region*). Estimation of the probable maximum storm may involve the study of hydrometeorological conditions and rainfall data of all the past major storms of the basin, and those of the region which probably could have occurred on the basin.

In order to compute the design storm of the given frequency, the study may involve developing of **IDF curve** (*intensity-duration-frequency curve*) for this design frequency. Such a curve will indicate the maximum rainfall depths during convenient unit time intervals, into which the total *critical storm duration*** may be subdivided, as depth = intensity × duration. These rainfall depths can then be applied successively with appropriate lags, on the derived unit hydrograph of the same unit duration (the selected time interval), so as to finally compute the flood hydrograph for the total rain storm.

When the design frequency is not considered and the flood hydrograph is to be computed for the *probable maximum precipitation* (PMP), such as for the design of spillways and in dam safety studies, the DAD curves will have to be developed. The *maximum observed DAD curves* of the region or of the basin shall usually be considered to obtain rainfall depths for the *standard project storm*; whereas the curves shall be

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* discussed in the chapter of "Flood Forecasting" in author's "Hydrology and Water Resources Engineering".

** The *duration of the critical rainfall* is usually taken to be equal to the basin lag, if the flood peak is of importance, but if the flood volume is of importance, then this duration is taken to be equal to the duration of the longest storm experienced in the basin.

further maximised to obtain the *absolute maxium DAD curves** for obtaining rainfall depths for the *probable maximum storm.* These DAD curves for different durations, such as 6 h, 12 h, 18 h, 24 h,..., shall thus be utilised to read out maximum rainfall depths (for the given basin area) such as 7 cm, 12 cm, 16 cm, 18 cm, ..., respectively. These comulative rainfall depths can be converted into *Excess rainfall depths* by subtracting the *initial loss* including the *infiltration* (generally, the minimum loss is considered, since the worst flood may enter at a time when the basin may be fully wet, with infiltration capacity at its minimum value). The excess rainfall depths produced in the successive unit intervals (6 h in the above case) are then applied on the unit hydrograph of that unit duration, to obtain the flood hydrograph for the *critical storm duration* of say 1 day, 2 days or 3 days. The excess rainfall increments during successive unit intervals should, however, be pre-arranged in a *critical sequence*, so as to produce maximum flood peak, when applied to the relevant unit hydrograph of the basin. This **critical sequence** can either be obtained by hit and trial, or the following practical procedure may be adopted.

The increments of excess rainfall are arranged opposite the ordinates of unit hydrograph is such a way that the largest excess rainfall increment is placed opposite the largest unit hydrograph ordinate; the second largest increment is placed opposite the second largest ordinate; and so on. This placement is finally reversed, so as to obtain the critical sequence, and the design storm.

The above procedure is illustrated in Example 7.61 below :

Example 7.61. *The projected cumulative rainfall depths for a 48-h design storm have been estimated in Table I below :*

Table I

Time (h)	*0*	*12*	*24*	*36*	*48*
Cumulative rainfall in cm	*0*	*10.2*	*30.5*	*34.0*	*36.0*

The 12 h unit hydrograph for the basin has also been drawn, and its ordinates are given in Table II below :

Table II

Time (h)	*0*	*12*	*24*	*36*	*48*	*60*	*72*	*84*	*96*	*108*	*120*	*132*
Ordinates of 12-h U.H. in m^3/s	*0*	*32*	*96*	*130*	*126*	*98*	*75*	*50*	*30*	*15*	*7*	*0*

Using the given rainfall and U.H. data, compute (i) the design storm hyetograph of excess rain; (ii) the design flood hydrograph for the catchment. Assume $\phi_{index} = 0.15$ cm/h and base flow of 50 m^3/s.

Solution. The design 48 h storm has to be computed from the given rainfall depths by determining the critical sequence of excess rainfall depths in 12 h intervals. These computations are carried out in Table 7.74 which is self explanatory. Values of excess

* These absolute maximum values are the estimated limiting values, which may occur only under extremely rare and worst combinations of the hydrometeoroiogical conditions in the basin. Their valuation is usually carried out by expert hydrometeorologists on the consideration of moisture actually contained in the observed peak storm, and that which a storm can possibly contain under very rare and worst combinations of rain causing factors.

rainfalls obtained in col (8) of this table, represent the design storm, a hyetograph for which is drawn in Fig. 7.129. **Ans.**

Table 7.74

Time (h)	*Cumulative rainfall, given (cm)*	*12-h incremental rainfall (cm)*	*ordinates of 6-h U.H. (m^3/s)*	*First arrangement of rainfall increments**	*Design sequence of rainfall increments***	*Infiltration loss @ 0.15 cm/h (cm)*	*Rainfall excess for the design storm in each interval of 12 h col (6) – col (7) (cm)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
0	0		0		0	0	0
12	10.2	*10.2*	32		3.5	1.8	1.7
24	30.5	**20.3**	96	2.0	10.2	1.8	8.4
36	34.0	3.5	**130**	**20.3**	20.3	1.8	18.5
48	36.0	2.0	*126*	*10.2*	2.0	1.8	0.2
60			98	3.5			
72			75				
84			50				
96			30				
108			15				
120			7				
132			0				

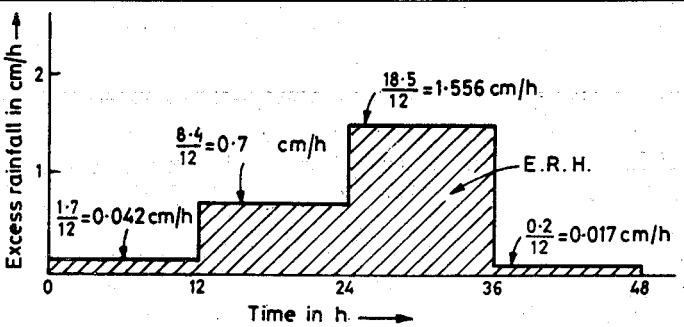


Fig. 7.129. Hyetograph of Excess rain for design storm in example 7.61.

To compute the design flood hydrograph corresponding to these excess rainfalls of 1.7, 8.4, 18.5 and 0.2 cm in successive 12 h intervals, the unit hydrograph ordinates are multiplied by the excess rainfalls and successively placed at lag of 12 h from each other, and totalled to compute the design DRH, to which is added the base flow to compute the design flood hydrograph. These calculations are carried out in Table 7.75; which is

*Peak value of rainfall increment (20.3 cm) is placed against the peak value of ordinate (130 m^3/s) and so on.

**Col. (6) is reversed sequence of col. (5).

self explanatory. The peak value of flood discharge comes out to 3699.2 m^3/sec and it occurs after 60 h from the start of the storm. **Ans.**

Table 7.75

Time from start of rain (1)	*12-h U.H. ordinates* (m^3/s) (2)	*Hydrograph ordinates produced by 1st 12-h effective rain of 1.7 cm = 1.7 × col. (2)* (3)	*Hydrograph ordinates produced by 2nd 12-h E.R. of 8.4 cm = 8.4 × col (2) at a lag of 12 h* (4)	*Hydrograph ordinates produced by 3rd 12-h E.R. of 18.5 cm = 18.5 × col (2) at a lag of 24 h* (5)	*Hydrograph ordinates produced by 4th 12-h E.R. of 0.2 cm = 0.2 × col (2) at a lag of 36 h* (6)	*D.R.H. ordinates; col (3) + (4) + (5) + (6), (m^3/s)* (7)	*Flood hydrograph ordinates col (6) + 50 (m^3/s)* (8)
0	0	0	—	—	—	0	50
12	32	54.4	0	—	—	54.4	104.4
24	96	163.2	268.8	0	—	432.0	482.0
36	130	221.0	806.4	592.0	0	1619.4	1669.4
48	126	214.2	1092.0	1776.0	6.4	3088.6	3138.6
60	98	166.6	1058.4	2405.0	19.2	3649.2	**3699.2**
72	75	127.5	823.2	2331.0	26.0	3307.7	3357.7
84	50	85.0	630.0	1813.0	25.2	2553.2	2603.2
96	30	51.0	420.0	1387.5	19.6	1878.1	1928.1
108	15	25.5	252.0	925.0	15.0	1217.5	1267.5
120	7	11.9	126.0	555.0	10.0	702.9	752.9
132	0	0	58.8	277.5	6.0	342.3	392.3
144	—	—	0	129.5	3.0	132.5	182.5
156	—	—	—	0	1.4	1.4	51.4
168	—	—	—	—	0	0	50

7.51. C.W.C. Recommendations for Choosing Design Flood Values for the Design of Hydraulic Structures

C.W.C. has recommended the adoption of the following design floods for various types of hydraulic structures :

(1) **For Spillways of Major and Medium Dams.** For spillways of major and medium dams, with storage capacity more than 6,000 hectare metres*, the *probable maximum flood* (PMF) should be chosen for the design flood.

The PMF should normally be calculated by unit hydrograph method for probable maximum storm. The *probable maximum storm* is an estimate of the physical upper limit to storm rainfall over a basin. It is obtained from the study of known storms of the region, and maximising them for moisture content of the rain producing air mass involved in the storms.

When unit hydrograph method cannot be used to compute the PMF due to very large catchment, and also, where long term peak annual discharge data is available, the Gumbel's probability method may be used to compute 1000 year frequency

* For such storages, generally, dam height is about 20 m or above.

flood, to arrive at a possible value of design flood. When no data is available, suitable empirical formula may be used to obtain a rough value of the design flood. However, the use of empirical formula or probability method, will give us only the peak design discharge, and not the complete design flood hydrograph; and hence, every effort should be made to collect the required hydrological data at the given site, so as to compute the full hydrograph of the design flood.

(2) **For Barrages and Minor Dams.** For design of barrages and minor dams with less than 6,000 hectare metres of storage, the *standard project flood* (SPF) or *100-year frequency flood*, whichever is more, may be adopted.

The standard project storm, to be used for computing standard project flood by unit hydrograph method, is generally taken as the largest storm, which has occurred in the region of the basin, during the period of weather records. It is not maximised for most critical atmospheric conditions, but it may be transposed from an adjacent region to the watershed under consideration, if needed.

CWC has also recommended that when the spillway capacity of such a dam being checked by impinging the above design flood in the reservoir over its full reservoir level, then some gate failures (maximum of 10% and minimum of 1 gate) may be considered, so that only the balance of the spillway crest is capable to pass the design flood discharge.

(3) **For Miscellaneous Hydraulic Structures.** For designing other hydraulic structures, such as weirs, aqueducts, cross-drainage works, flood embankments or levees, etc., 50 to 100-year frequency flood is to be normally adopted, though variations are possible and lesser or higher floods may be accepted for design, if the local conditions and the general discretion and common sense of the designer, can make the competent authority agreed to his recorded views.

PROBLEMS

1. (*a*) Define Hydrology and discuss critically the statement "Thorough knowledge of hydrology is a must for any water resources planning"?

(*b*) What is "hydrologic cycle" and what is its importance ?

2. Explain as to how weather affects precipitation, and precipitation in turn affects the design of water resources projects.

3. Discuss as to why the study of meteorology is important to Hydrologists, and how the new developments in meteorology may enrich hydrology ?

4. Define precipitation, indicating its various types. Also discuss the role of weather and clouds in causing precipitation.

5. (*a*) Define the terms : (*i*) temperature lapse rate ; and (*ii*) dew point.

(*b*) Discuss the three important ways in which clouds are formed indicating the type of precipitation caused by each of them.

6. (*a*) Enumerate the basic forms of clouds, and their sub divisions. Which of them have the high potential for precipitation ?

(*b*) How are clouds measured in the sky ?

(*c*) Discuss as to why and when the clouds lead to precipitation.

7. Some clouds cause higher rain, some cause heavier rain, and some others cause snowfall. In this context, discuss critically the formation of rain drops and snowflakes in clouds.

8. Distinguish between 'cold clouds' and 'warm clouds', explaining simultaneously the process of formation of rain drops in each of them.

9. (*a*) Distinguish between 'short range' and 'long range' weather forecasts.

(*b*) Discuss briefly the use of Radars and Satellites in weather detection and forecasting.

10. (*a*) Define and explain the term 'meteors' and 'hydrometeors', indicating their constitute phenomena.

(*b*) Critically discuss as to how a radar helps in checking and supplementing rain gauge data.

(*c*) What are the limitations of radars in weather probes.

11. (*a*) What is a geosynchronous satellite, and what are its important constituent systems ? Discuss briefly the functions performed by each constituent system.

(*b*) Write a brief note on INSAT-2A.

12. Discuss INSAT-programme and achievements made so far. What kind of services and uses are being performed by INSATs.

13. Write notes on :

(*i*) Orographic and turbulent ascent in precipitations.

(*ii*) Lapse rates.

(*iii*) Satellite imageries.

(*iv*) Role of INSAT-1B and INSAT-2A in disaster management in cyclone prone coastal areas.

14. Define 'Precipitation'. What is cyclonic precipitation? Distinguish between cold front and warm front. Explain how you would estimate missing precipitation data.

15. (*a*) What is meant by rain-gauges'. and what is their use?

(*b*) Enumerate the different types of rain gauges, and describe with a neat sketch, the construction and functioning of the non-recording type of gauge being used in India.

16. (*a*) Differentiate between Recording and Non-recording types of raingauges.

(*b*) Describe with a neat sketch the construction and use of a float-type of a recording gauge.

(*c*) Which type of gauge is used for remote recording of rainfall?

[**Ans.** Tipping bucket type]

17. (*a*) How does rainfall measured in India?

(*b*) At what time the rainfall is recorded in Delhi. [**Ans.** 8.30 A.M. Daily]

(c) Discuss with a neat sketch the construction and use of a 'tipping bucket type' of a recording gauge.

18. Define and explain the following terms:

(*i*) Catchment area. (*ii*) Runoff and Surface runoff.

(*iii*) Hydrograph of a storm (*iv*) Yield of a drainage basin.

(*v*) Average annual rainfall (*vi*) Index of wetness.

19. (*a*) Discuss briefly the three important methods, which are used for determining the mean rainfall over a drainage basin.

(*b*) Find the mean precipitation for the area sketched below by Thiessen's method. The area is

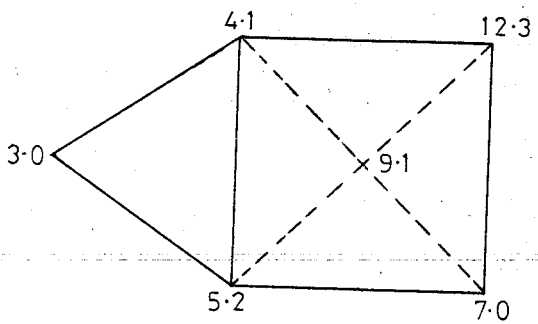


Fig. 7.130

composed of a square plus an equilateral triangular plot of side 2.5 kilometres. Rainfall readings in cm at the various stations are given in the figure.

[**Hint:** Follow example 7.4, and determine mean rainfall = 6.91 cm. **Ans.**]

20. Explain the following terms:

(*a*) cyclone; (*b*) tornado; (*c*) orographic lifting; (*d*) index of wetness; and (*e*) rain gauge density.

Mention the different types of rain gauges used, and state their merits and demerits. Explain how rain gauge readings get affected with varying altitude.

Estimate the true amount of rainfall, if a catch of 80 cm is made by a gauge whose height is (*i*) 4.8 m, and (*ii*) 24.3 m.

21. (*a*) Explain with a sketch Thiessen's method for calculating average depth of precipitation over an area. Discuss the relative merits of this method over other methods of computation.

(*b*) The isohyets for annual-rainfall over a catchment were drawn. The areas of strips between the isohyets are indicated below. Find the average depth of annual precipitation over the basin.

Isohyets (cm)	*Area between Isohyets (sq km)*
9—10	22
10—11	80
11—12	105
12—13	98
13—14	78
14—15	16

[**Hint :** Follow example 7.5] [**Ans.** 11.95 cm]

22. Describe in detail how a rain-gauge is set up over a place to measure the rainfall of that place. What are the requirements of the rain gauge? What are the errors likely to take place during measurements?

How many times in a day and at what times is the precipitation measured and recorded in Tamil Nadu?

23. Explain the three different methods employed to determine the mean precipitation over a given area. Find the mean precipitation for a rectangular area given with precipitations marked in figure, by any two of the above methods.

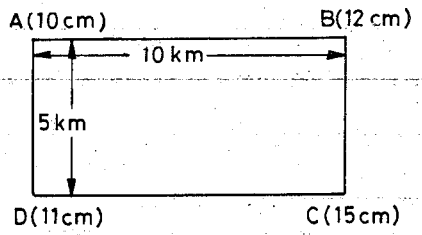


Fig. 7.131

[**Ans.** 12 cm by arithmetic as well as by Thiessen's methods)

24. Explain the different types of precipitation with examples. Can we use Radar as an aid in the measurements of rainfall? How?

Explain with a graph, the percentage deviation from the average annual precipitation with duration of observations in years.

Explain how you can supplement precipitation records if they are missing for a particular year.

25. Draw a neat sketch of an automatic recording rain gauge and recording of the rainfall measurements?

Describe the Thiessen-polygon method of computing average rainfall over an area.

26. (*a*) Explain:

(*i*) Hydrologic failure (*ii*) Frontal surface.

(*iii*) Meteorological homogeneity (*iv*) Lapse rate.

(*b*) Fig. 7.132, represents a catchment area with the precipitations observed in a year. Calculate the

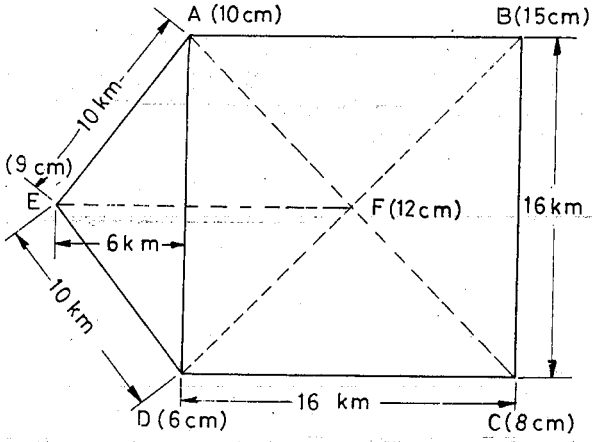


Fig. 7.132

mean precipitation by Thiessen method, and check the result roughly by the arithmetic mean method.

[**Ans,** 10.47 cm by Thiessen's method; and 10.00 cm by arithmetic mean.]

27. (*a*) What is meant by rain gauge density? What are the standards prescribed for it for different regions?

(*b*) How would you determine statistically, the optimum number of rain gauges required to be installed in a given catchment.

(*c*) In a certain river basin, there are four raingauge stations, with their normal annual precipitations amounting to 800, 520, 440 and 400 mm, respectively. Determine the optimum number of rain gauges in the catchment, if it is desired to limit the error in the mean value of rainfall in the catchment to 12%

[**Hint:** Follow example 7.3] [**Ans.** 8 Nos,]

28. (*a*) What is meant by recurrence interval of a storm?

(*b*) Discuss briefly the station-year method used for estimation of recurrence interval of precipitation. What are the limitations of the method.

(*c*) Explain by a sketch, how frequency data for storms of various durations can be represented by intensity-duration curves.

(*d*) The values of annual precipitations at a rain gauge station expressed in cm per year in chronological sequence from 1967 to 1976 are indicated below:

70.2	31.7	68.8	50.9	60.3
58.7	43.9	50.3	48.7	67.5

Using Hazen's or any other suitable method, estimate the value of precipitation, which has a recurrence interval of 5 year.

(**Hint :** Follow example 7.9). [**Ans.** 68.8]

29. What is an intensity-duration curve, and how will you proceed to derive such a curve for a given frequency at a rain gauge station from the available data of worst storms of different durations kept for a sufficient number of years.

(*b*) Write down the general expression for intensity-duration relationship of rainfall. Explain the necessity for frequency analysis.

30. (*a*) Estimate by the normal ratio method, the rainfall at station *A* in the year 1958, using the following data for the stations *A,B,C,D,* and *E.*

Year	Annual Rainfall in cm				
	A	B	C	D	E
1958	—	96.5	99.3	91.6	88.4
1959	110.9	108.0	110.0	102.2	101.7
1960	108.1	105.8	107.7	99.5	95.6
1961	96.66	96.9	98.4	78.5	71.1
1962	101.4	101.4	116.4	83.2	111.3
1963	103.8	98.9	96.5	81.9	81.4
1964	89.7	89.0	85.9	77.0	89.8
1965	149.0	143.1	141.1	120.9	124.4

(*b*) Check the consistency of rainfall records of station *B*. Adjust, if necessary, the rainfall values by the double mass curve technique.

31. (*a*) Explain briefly the rainfall-runoff process, indicating all the possible losses involved.

(*b*) Explain clearly the following terms :

(*i*) Rainfall interception (*ii*) Inflitration capacity

(*iii*) Infiltration rate (*iv*) Rainfall excess

(*v*) Residual rain

32. Differentiate between

(*i*) runoff and surface runoff

(*ii*) runoff and yield of a drainage basin.

33. (*a*) Explain clearly the three different constituents of runoff.

(*b*) What is a hydrograph, and what is its use in hydrology ?

34. Differentiate between 'influent' and 'effluent' streams. Also explain with a suitable sketch as to how river can behave both as an influent as well as an effluent stream. Also explain the term bank storage in this connection.

35. (*a*) What is meant by 'ground water depletion curve' and how it is derived ?

(*b*) How will you proceed to separate the base flow from a hydrograph of a river discharge ?

36. What is evaporation? Mention the factors controlling the evaporation process. What do you understand by water budget determination of reservoir evaporation?

Discuss critically the use of auxiliary pan measurements for the determination of evaporation from water surface.

37. (*a*) Explain the difference between evaporation, interception and transpiration. What is transpiration ratio?

(*b*) Discuss Dalton's law for evaporation from water surfaces. For a large water surface, explain the influence of depth on the rate of evaporation.

38. Explain: (*a*) Evaporation, (*b*) Infiltration, (*c*) Interception, (*d*) Transpiration, (*e*) Percolation, and (*f*) Consumptive use.

Discuss the need for the conservation of available supplies of water and describe the methods adopted for evaporation control from reservoirs.

39. Briefly describe any method by which you can measure 'the evaporation loss' from a free-water surface.

Explain any two methods of reducing the evaporation loss from a free-water surface.

40. (*a*) Explain briefly the method of measuring evaporation in a certain area.

(*b*) Write short notes on:

(*i*) Transpiration and Transpiration ratio

(*ii*) Evaporation opportunity

(*iii*) Water-shed leakage.

41. Write down the most common empirical formula used to calculate evaporation?

What are the factors influencing evaporation?

Describe any one practical method of determining evaporation.

42. (*a*) What are the principal types of installations that can be used in a stream to obtain continuous records of discharge measurements?

(*b*) The following data is observed on a stream, in a standard current meter test:

Distance from right bank (m)	0	2	4	6	9	12	15	18	20	22	23	24
Depth (m)	0	0.5	1.10	1.95	2.25	1.85	1.75	1.65	1.50	1.25	0.75	0
No. of revolutions at 0.6 depth	0	80	83	131	139	121	114	109	92	85	70	0
Time (s)	0	180	120	120	120	120	120	120	120	120	120	0

The rating equation of the current meter is

$V = 0.32N + 0.032$ m/s, where, N is revolutions/sec. Calculate the discharge of the stream, using mid-section method. [**Ans.** 11.73 cumecs]

43. (*a*) What is a rating curve? Explain its use. Sketch a typical rating curve.

(*b*) Name the different methods, which can be applied for extending a rating curve, and the circumstances under which a particular method is to be adopted. Explain briefly any one of these methods.

44. (*a*) What is a stage discharge curve ? How is it affected by a changing stage of the river compared to a constant stage ?

(*b*) What is meant by a 'permanent control' and a 'shifting control'? How will you detect the possibility of a shifting control? What remedies do you suggest to obtain correct gauge measurements when the control is shifting (*i*) slowly and only during floods, (*ii*) quite rapidly and constantly?

45. Discuss briefly the different methods by which peak discharge of a stream during a storm can be estimated, when it may not be possible to take current-meter measurements.

During a flood, the depth of water in a rectangular channel, 10 metres wide was found to decrease downstream from 3.0 to 2.94 m over a reach of 90 m. The bottom slope of the channel was 0.01 and Manning's n was 0.025. Estimate the flood discharge through the channel.

46. (*a*) What is a current-meter and what are its types? Discuss how can it be used for determining the discharge (*i*) in a shallow river; (*ii*) in a wider river. What formulas are used for computing discharge measurements.

(*b*) Explain briefly the 'Stout's method' for correcting gauge discharge measurements.

(*c*) Explain a method, for extending a 'Rating curve', which can be applied irrespective of the shape of the river cross-section.

47. What is meant by 'stream gauging'? Describe the velocity area method that is used for stream gauging.

48. Enumerate the different methods which are used for stream gauging. Discuss any one of these methods in details.

49. (*a*) Enumerate the different methods of measuring discharge.

(*b*) How will you measure the discharge of:

(*i*) a river?

(*ii*) a small stream?

(*iii*) a canal?

[**Hint.** (*i*) use velocity area method;
(*ii*) use weir method;
(*iii*) use venturi-flume.]

50. (*a*) Fill in the blanks with appropriate words.

(*i*) Stream gauging helps in determining the ..... of a catchment. (runoff/characteristics)

(*ii*) A current meter is used to measure the ..... of flow of water. (depth/velocity)

(*iii*) Venturi flume is used to measure the discharge of a ..... (river/canal)

[**Ans.** (*i*) runoff; (*ii*) velocity; (*iii*) canal)]

(*b*) Describe briefly the dilution technique of streamflow measurement.

51. What is meant by 'Runoff' and how is it produced? Discuss briefly the various factors on which it depends.

52. Define and explain the following :

(*i*) Time of concentration

(*ii*) Period of surface runoff

(*iii*) Initial basin recharge.

53. (*a*) What is meant by 'runoff'? How is it produced ? What is the difference between runoff and surface runoff?

(*b*) Fill in the blanks with appropriate words.

(*i*) Runoff is measured in..... (cu.m., cu.m. per sec., m. per sec.)

(*ii*) The runoff will be more if the catchment is .....shaped. (fan/fern)

[**Ans.** (*i*) cu.m. per sec; (*ii*) fan]

54. Enumerate and briefly discuss the various factors which affect the runoff from a catchment.

55. (*a*) What is meant by 'runoff' and how does it produced ? Discuss the various factors on which it depends.

(*b*) Define and explain the following :

(*i*) Time of concentration

(*ii*) Period of surface runoff

(*iii*) Initial basin recharge.

56. Explain, with sketches if necessary, the following methods for calculating net storm rain :

(*a*) Infiltration capacity method ;

(*b*) ϕ_{index} method : and

(*c*) W_{index} method.

[**Hint.** Net storm rain means runoff]

A three hour storm occurs over a 62.0 sq. km area. From the following data, estimate the net storm rain for the whole area and its hourly distribution :

Sub-area (sq km)	ϕ_{index} *cm/hr*	*Hourly rain*		
		1st	*2nd*	*3rd*
14.0	2.0	1.5	5.0	0.8
20.0	3.5	1.5	5.0	0.8
28.0	1.0	1.6	5.5	

57. (*a*) State and explain the 'Rational method', which is used for computing the peak rate of runoff for the design of urban storm water drains. What are the limitations of this method ?

(*b*) How will you compute the concentration time of a small catchment having flow channels in it?

58. Mention the basic assumptions in the theory of unit hydrograph. Explain step by step the method of construction of unit hydrograph from a storm of unit duration. Mention the sources of error in unit hydrograph construction.

What is a distribution graph ? If a unit hydrograph for a storm of particular duration is available, how would you construct a unit graph for a storm of another duration ?

59. Explain briefly what a unit hydrograph and a distribution graph is ?

Starting from 12 noon, storm rainfalls of 2.5, 7.5 and 5.0 cm occur during three successive hours over a 25 square kilometre area. The storm loss rate (ϕ_{index}) is 1.25 cm per hour. The percentages of

distribution graph for successive hours are 5, 20, 40, 20, 10 and 5. Estimate the value of peak discharge in cubic metres per second and the hour when it is expected.

60. Analysis of the runoff records for unit storm over a catchment basin yields the following data:

Observed surface discharge at the concentration point on successive days are 19.6, 62.4, 151.3, 133.0, 89.5, 63.1, 43.5, 28.6 and 19.6 cumecs.

Estimated base flow during the corresponding period on successive days are 19.6, 22.4, 25.3, 28.0, 25.6, 22.5 and 19.6 cumecs.

Determine the distribution graph percentages.

On the same catchment basin (area = 2850 sq. km), there were rainfalls of 7 cm per day on July 15, and 10 cm per day on July 18 of a certain year. Assuming a storm loss rate of 2 cm per day (ϕ_{index}), estimate the value of peak surface runoff in m^3/sec and the date of its occurrence.

61. What is a 'Unit hydrograph' ? How would you obtain a Unit hydrograph from an isolated intense short duration storm occuring uniformly over the basin ?

The ordinates of a 6 hour Unit hydrograph are as under :

Time (from beginning of rainfall) in hours	*Ordinate of the Unit Hydrograph in cumecs*
0	0
6	20
12	50
18	150
24	120
30	90
36	70
42	50
48	30
54	20
60	10
66	0

If two storms, each of unit rainfall excess in 6 hours duration, reach the catchment in succession, then draw the hydrograph resulting from these two storms. The stream may be assumed to have a uniform base flow of 2 cumecs.

62. Explain the method by which maximum runoff can be estimated from a catchment.

Describe the method of computing the ordinates of a Unit-hydrograph from the data of a flood hydrograph.

63. Define 'Unit hydrograph'. State its uses.

The following are the observations of a flood hydrograph from a drainage of 1700 square kilometres due to a 12 hour rainfall (base flow has been subtracted):

Time from beginning in days	*Discharge in cumecs*	*Time from beginning in days*	*Discharge in cumecs*
0	0	4.0	360
0.5	30	4.5	275
1.0	100	5.0	180
1.5	365	5.5	125
2.0	645	6.0	85
2.5	700	6.5	50
3.0	585	7.0	25
3.5	475	7.5	0

Derive the ordinates of 12 hour unit hydrograph.

64. An isolated intense storm of 30 minutes duration occurred over a catchment basin of 283.28 hectares. The estimated average depth of precipitation was 6.35 cm over the entire area, and half hourly streamflows resulting from the above storm were noted as follows :

Time	*Q in cumecs*	*Time*	*Q in cumecs*
7.00 a.m.	3.40	1.00 p.m.	7.93
7.30 a.m.	3.35	1.30 p.m.	7.20
8.00 a.m.	3.30	2.00 p.m.	5.60
8.30 a.m.	4.00	2.30 p.m.	5.30
9.00 a.m.	6.40	3.00 p.m.	4.60
9.30 a.m.	6.90	3.30 p.m.	—
10.00 a.m.	13.75	4.00 p.m.	4.40
10.30 a.m.	13.45	4.30 p.m.	3.85
11.00 a.m.	13.30	5.00 p.m.	3.75
11.30 a.m.	11.05	5.30 p.m.	3.70
12.00 noon	9.65	6.00 p.m.	—
12.30 p.m.	6.78	6.30 p.m.	3.70

Draw the unit hydrograph for the above basin.

65. (*a*) What is a unit hydrograph ? Discuss its uses, and explain how a 6-hour unit hydrograph can be determined from a given 3-hour unit hydrograph.

(*b*) In a certain basin, ordinates of a unit hydrograph (1 cm–6 hr) are given below :

Time (in hr)	0	6	12	18	24	30	36	42	48
Ordinates (cumecs)	0	4	12	25	18	12	7	4	0

Determine the peak flood and the total volume of flood flow in the basin corresponding to storm described below :

Period (h)	0—6	6—12
Runoff (mm)	10	20

Base flow at the time of storm was 5 cumecs.

66. (*a*) Define a 'Unit hydrograph', and explain how is it used to estimate the flood hydrograph of a storm of a given magnitude and of the same duration.

(*b*) What is a *S*-curve hydrograph ? How is it constructed, and what is it used for ?

67. (*a*) What is meant by 'Design Flood', and what is its importance?

(*b*) Enumerate the various methods which can be used for estimating design flood discharge from a certain catchment, and discuss one of these methods in details.

68. What are the various methods of estimating the design flood of a catchment.

Explain how Dicken originally worked his empirical flood formula for calculating the flood discharge of a catchment.

What are the flood coefficients used in the above formula for different places in India?

69. Describe briefly the various methods for obtaining the maximum flood discharge of a river.

Find the values of C and n in the empirical formula $Q = C A^n$ for flood discharge Q in m^3/sec from a catchment of area A km^2 from the following data:

A, km^2	4	10	50	100	200
Q, m^3/sec	100	158	354	500	706

70. (*a*) "The results obtained by the probability methods for determining the design flood of 100 years frequency with 10 years past data cannot be relied upon." Discuss critically the above statement. Also discuss the modern unit hydrograph method (which has substituted the use of the probability methods) for determining design flood flows in such cases.

(*b*) Find out the frequency of a flood of magnitude 10,000 cumecs, given the following record of maximum yearly peak floods for 10 years. Also discuss the preciseness of the result, so obtained.

Year	*Flood peak in cumecs*
1951	3200
1952	4250
1953	6250
1954	3100
1955	2800
1656	3500
1957	8500
1958	8900
1959	4200
1960	5200

8

Rivers, Their Behaviour, Control and Training

8.1. Importance of Rivers and Necessity of Controlling Them

Water is the most important natural resource not only of a state or of a country, but of the entire humanity. The prosperity of a nation depends primarily upon the exploitation of this resource. Thus, it can be stated that the primary wealth of a nation is water, which flows in rivers and streams. This itself establishes the importance of rivers, and no other explanation is required to stress their importance.

Importance of rivers has been recognised since time immemorial, and rivers have occupied a very prominent place in every stage of human development. With the incoming of modernisation and mechanisation of human life, water and hence, rivers are becoming more and more important. They provide us water for industrial uses and are a source of cheap energy. Rivers have always been satisfying our domestic, municipal, irrigation, and other demands, and that is why most of our cities were established in the vicinity of the natural rivers. In the primitive times, there was absolutely no control on these natural rivers, and hence, they used to cause tremendous devastations and troubles to human beings. But with the development of science and technology, and in his bid to control nature, man has devised and is devising means and ways to control these mighty rivers. In the above context, we have to study the rivers, their behaviour, and to develop means and ways, so as to ensure an effective control upon them.

Rivers take off from mountains, flow from the mountainous plain terrains, and finally join the oceans. They are formed along more or less defined channels, drain away the land water obtained by precipitation and snow melting in high altitudes, and discharge the unutilised waters back into the sea, thus completing the hydrological cycle. The rivers not only carry this huge amount of water but also carry a tremendous amount of silt and sediment which is washed down from the catchment area or gets eroded from their beds and banks. Before we discuss the techniques that are often adopted for controlling and improving the rivers, let us, first of all, discuss the various types of rivers and their characteristics.

8.2. Types of Rivers and Their Characteristics

8.2.1. Classification of Rivers on the Basis of the Topography of the River Basin. Depending upon the topography of the basin, the river reaches can be classified into two main classes, *i.e.*

(*i*) *Rivers in hills* (Upper reaches) ;

(*ii*) *Rivers in alluvial plains*, known as rivers in flood plains (Lower reaches) ; and

(*iii*) *Tidal rivers*

All these three types of river reaches are described below :

(*i*) **Rivers in Hills (Upper Reaches)**. The rivers generally take off from the mountains and flow through the hilly regions before traversing the plains. These upper reaches of the rivers may be termed as *Rivers in Hills*. They can be further sub-divided into :

(*a*) Incised or Rocky River stage ; and

(*b*) Boulder River stage.

(*a*) *Rocky Stage or Incised River Stage (Upper reaches)*. In this type, the flow channel is generally formed by the process of degradation (erosion). The sediment transported in this reach is often different from the river bed material, since most of it comes from the catchment due to denundation and soil erosion. These river-reaches are highly steep with swift flow, and forming rapids along their courses. The beds and banks of such rivers are less susceptible to erosion. The bed-load carried by such river-reaches cannot be determined on the basis of usual bed load transportation formulas, derived on the basis of bed characteristics.

(*b*) *Boulder River Stage (Upper reaches)*. The river bed in these reaches consists of a mixture of boulders, gravels, shingles and alluvial sand-deposits created by itself. Still these river reaches differ considerably from those carrying sand and silt and flowing through plains. In the latter stage, the river flows through deep well defined beds and wider flood plains and develops zig-zag courses. On the other hand, in the boulder stage, *the river flows through wide shallow beds and interlaced channels, and develops a straighter course*. During a flood, the boulders, shingles and gravels are transported downstream, but as the flood subsides, the material gets deposited in heaps. The water, then unable to shift these heaps, go round them, and the channel often wanders in new directions, often attacking the banks and consequently widening the bed.

(*ii*) **Rivers in Alluvial Flood Plains (Lower Reaches)**. The chief characteristics of these river reaches is the zig-zag fashion in which they flow, called **meandering**. They meander freely from one bank to another and carry sediment which is similar to bed material. Material gets eroded constantly from the concave bank (outer edge) of the bend and gets deposited either on the convex side (inner edge) of the successive bends or between two successive bends to form a bar, as shown in Fig. 8.1.

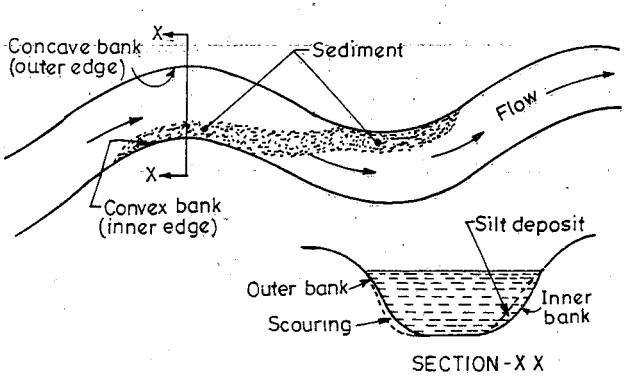


Fig. 8.1. Silting and scouring in a meandering river.

When once a straight moving river, just slightly deviates from its axis, the unbalance created goes on multiplying *with constant erosion from the concave side and deposition on the convex side.* If unchecked, the process continues, resulting in the formation of large meanders.

Rivers in flood plains can be further classified as :

(*a*) *Aggrading* ; (*b*) *Degrading* ; (*c*) *Stable* ; (*d*) *Braided* ; and (*e*) *Deltaic.*

If the river is collecting sediment and is building up its bed, it is called an *aggrading* or of an *accreting type*. If the bed is getting scoured year to year, it is called a *degrading type*. If there is no silting or scouring, it is called a *stable river*. It is not necessary that a river reach should be of one type in its entire alluvial length, rather it is generally of more than one type of reach, in its length. In other words, the same river reach may behave as of aggrading or of degrading or of stable type. How and under what circumstances, the river may change its type, will become very clear, if we study these types in details, as given below :

(*a*) *Aggrading or Accreting Type.* An **aggrading river** is a silting river, as shown in Fig. 8.2. Such a river increases its bed slope, which is called *building up of slope*. The silting may be due to various reasons, such as heavy sediment load; construction of an obstruction across the river, such as a dam or a weir; sudden intrusion of sediment from a tributary; etc. This type of river, usually, has straight and wide reaches with shoals in the middle, which shift with floods, dividing the flow into a number of braided channels.

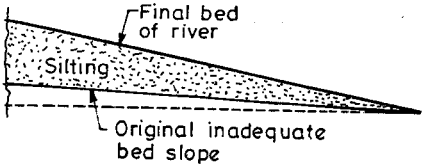


Fig. 8.2. Aggrading River

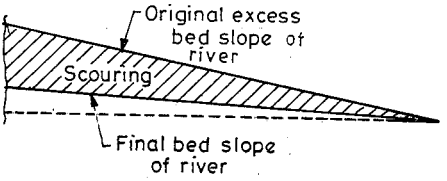


Fig. 8.3. Degrading River.

(*b*) *Degrading Type.* If the river bed is constanly getting scoured (eroded) to reduce and dissipate available excess land slope, as shown in Fig. 8.3, then the river is known as a **degrading river.** It may be found either above a *cutoff** or below a dam or a weir or a barrage, etc. For example, the Colorado River (U.S.A.) became a degrading type on the downstream, after the construction of the Boulder dam.

(*c*) *Stable Type.* A river which does not change its alignment, slope and its regime significantly, is called a *stable river.* Changes such as silting or scouring or advancement of delta into the sea may take place, but they are negligible and may fail to produce any change in the regime of the channel, except, perhaps, that the river may shift within its *Khadirs***. Most of the sediment load carried by them is brought to the sea.

The behaviour of a particular reach (whether to be aggrading, degrading or stable) depends mainly upon the variations of silt charge (size as well as quantity) and flow discharge with time.

(*d*) *Braided Rivers.* When a river flowe in two or more channels around alluvial islands (as shown in Fig. 8.4), it is known as a **braided river.**

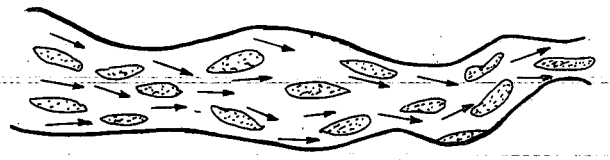


Fig. 8.4. A typical braided reach of a river.

The braided pattern in a river develops after local deposition of coarser material, which cannot be transported under prevailing conditions of flow, and which subsequently grows into an island consisting of coarse as well as fine material.

* Explained later.

** The extreme limits within which a river is known to have ever wandered.

(*e*) *Deltaic Rivers.* A river before it joins the sea, gets divided into branches, thus forming a Δ shaped delta, as shown in Fig. 8.5.

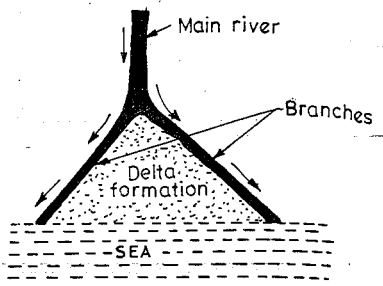


Fig. 8.5. Delta formation.

As the river approaches the sea, its velocity is reduced, and consequently the channel gets silted and water level rises resulting in spills and eventual formations of new channels. These branches multiply in their number as the river approaches the sea. Causes of delta formation and factors responsible for its shape and growth etc., are beyond the scope of this book. The **delta river** indicates a stage, rather than a type of a river.

(*iii*) **Tidal Rivers**. The tail reaches of the rivers adjoining the oceans are affected by the tides in the ocean. The ocean water enters the river during the flood tide and goes out into the ocean during the ebb tide. The river, therefore, undergoes periodical rise and fall in its water level, depending upon the nature of the tide. The distance upto which the tidal effect is experienced, depends upon various factors, such as the shape and configuration of the river, the tidal range, freshet discharge, etc. The detailed description of tidal reaches is beyond the scope of this book.

8.2.2. Classification of Rivers on the Basis of Flood Hydrographs. The rivers may be classified on the basis of stage and nature of their flood hydrograph, into the following two types :

(*i*) Flashy rivers ; and (*ii*) Virgin rivers.

(*i*) **Flashy Rivers.** If the flood rise and flood fall in a river is sudden, then it is called a *flashy river.* In flashy rivers, the flood flows, therefore, occur suddenly, and rise and fall of water level is very quick. The flood hydrographs are very steep, indicating floods, all of a sudden.

(*ii*) **Virgin Rivers.** In arid zones (deserts), a river water may completely dry before it joins another river or the ocean. Such a river is called a *virgin river.* After flowing for a certain distance from its source, the water of such a river disappears due to high percolation or due to excessive evaporation. There are several virgin rivers in the States of Kutch and Rajasthan in India.

8.3. Indian Rivers and Their Classification

Indian rivers can be broadly classified into :

(*i*) *Himalayan rivers* ; and

(*ii*) *Non-Himalayan rivers.*

(*i*) **Himalayan Rivers**. These rivers take off from Himalayas and flow through alluvial plains. They derive their water from rains during monsoon and winter, and from the melting snow during summer. These rivers are, therefore, almost perennial and can give dependable yields throughout the year. These rivers carry huge sediment, because of two reasons : (*i*) The Himalayan rocks are soft and friable (*ii*) The Himalayan zone, particularly the north-eastern part, is susceptible to earthquake disturbances, causing landslides and increased rock sediment. The important Himalayan rivers are

Indus, Jhelum, Chinab, Ravi, Beas, Sutlej, Ganga, Gandak, Ghaghar, Gomti, Kosi, Brahmaputra, etc.

Due to the heavy rainfall in the month of July and August (causing about $\frac{3}{4}$th of average annual rainfall), these North Indian rivers rise in high floods. The surface runoffs so caused are usually much more than the normal average flow of the entire year. The flood discharge may be as great as 50 to 100 times the normal winter flow. The sections required for passing flood discharges are vastly out of proportion to the sections required for normal winter flows. These large variations in discharge and sediment load make the hydraulics of these rivers very complex and cause them to meander.

(*ii*) **Non-Himalayan Rivers.** They are non-perennial rivers. They receive their major water supply only in rainy seasons ; and for the rest of the year, they may draw water from ground water as base flow. They flow in Central and South India and takeoff from Aravali, Vindhya and Satpura mountain ranges. *Chambal, Mahanadi, Cauvery, Godavari, Tapti, Narmada, etc.,* are the important examples of these rivers. These rivers are much more stable than the Himalayan rivers, and pose lesser problems, as they flow through non-alluvial soils. The dividing line between Himalayan & Non-Himalayan rivers is the boundary hetched by the left bank of river *Sutlej*, Right bank of river *Yamuna* and *Ganga* (after Allahabad) and left bank of river *Brahmaputra*, as can be seen in the map of Indian rivers, shown in the enclosed coloured photo Fig. 8.1.

Note : For regional classifications of Indian rivers, please refer article 1.5 in "Hydrology and Water Resources Engineering" by the same author.

8.4. Behaviour of Rivers

The chief factor which is responsible for moulding the behaviour of rivers is the silt and sediment that flows in the river. The sediment carried by the river poses numerous problems, such as : increasing of flood levels, silting of reservoirs, silting of irrigation and navigation channels, meandering of rivers, splitting up of a river into a number of interlaced channels, etc. The *meandering* causes the rivers to leave their original courses, forces them to flow along new courses, and thus devastating vast areas of land and affecting important and valuable nearby structures, such as bridges, railway lines, roads, etc. We shall now discuss the behaviour of the rivers in special situations, such as in bends, meanders and cutoffs, in details.

8.4.1. Straight Reaches. In a straight reach of a river, the river cross-section is in the shape of a trough, with high velocity flow in the middle of the section.

Since the velocity is higher in the middle, the water surface level will be lower in the middle and higher at the edges, as shown in Fig. 8.6. Due to the existence of this transverse gradient from sides towards the centre, transverse rotary currents get developed, as shown in Fig. 8.6. However, straight reaches are very few in alluvial rivers.

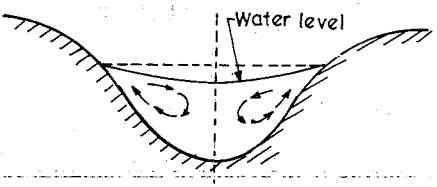


Fig. 8.6

8.4.2. Bends. Every alluvial river tends to develop bends, which are characterised by scouring on the concave side and silting on the convex side, as shown in Fig. 8.7. The silting and scouring in a bend may continue due to the action of the centrifugal force.

When the flow moves round a bend, a centrifugal force is exerted upon the water, which results in the formation of transverse slope of water surface from the convex edge to the concave edge, creating greater pressure near the convex edge (see Fig. 8.7). To keep its own level, water tends to move from the convex-side towards the concave side.

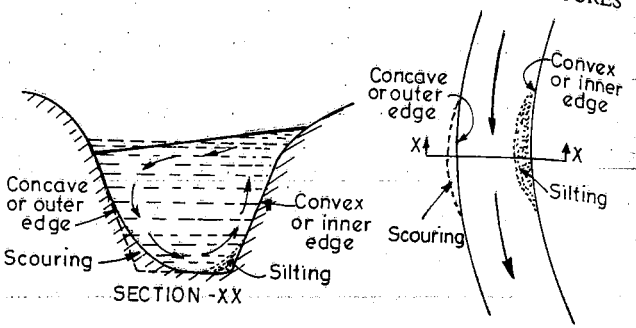


Fig. 8.7

However, the topmost water surface movement is prevented by the centrifugal force. Moreover, towards the bottom, the velocities are much less than towards the top, and enough centrifugal force is not available to counteract the tendency of water at the top to move inwards. Hence, the water dives in, from the top at the concave end, and moves at the bottom towards the convex end. These rotary currents cause the erosion of concave edge and deposition on convex edge, forming shoals on the convex sides. When once a bend is formed, the flow tends to make the curvature larger and larger.

8.4.3. Meanders. When once a river deviates from its axial path and a curvature is developed (either due to its own characteristics or due to the impressed external forces), the process moves downstream by building up shoals on the convex side by means of secondary currents. The formation of shoals on the convex side, results in further shifting of the outer bank by erosion on the concave side.

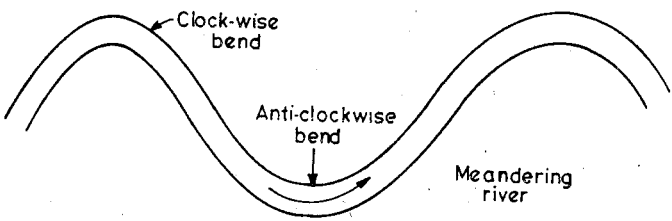


Fig. 8.8. Meandering River.

Formation of successive bends of reverse order may lead to the formation of a complete S curve called meander. When consecutive curves of reverse order connected with short straight reaches called (crossings) are developed in a river reach, the river is stated to be a meandering river (Fig. 8.8). In order to study the behaviour of a meandering river, the river may be supposed to follow a *sine curve.*

Causes of meandering. There was a belief that meandering is caused due to the presence of an excessive bed slope in the river that the excess energy is dissipated by increasing the channel length by meandering. The above theory has been proved to be incorrect.

The latest and widely accepted theory behind meandering is based upon the extra turbulence generated by the excess of river sediment during floods. During floods, the river carries tremendous amount of silt charge. *It has been established that when the silt charge is in excess of the quantity required for stability, the river starts building up its slope by depositing the silt on its bed. In other words, the river reach becomes an aggrading or of an accreting type. This accretion, is the primary process, which consequently leads to meandering, as explained below :*

The increase in river slope tends to increase the width of the channel, if the banks are not resistant. The banks are, thus, attacked by water, and in the process, one bank is likely to be attacked slightly more than the other, causing a slight deviation of flow. This slight deviation from uniform axial flow, *helps in moving more and more flow towards one bank than towards the other.* The process continues with more and more

vigour, causing more and more flow towards the former bank and forming shoals along the latter, thus, accentuating the curvature of flow, and finally, producing meanders in its wake.

The concave bank goes on eroding and the convex bank goes on silting. Further, it has been established that collection of sediment on the convex side is independent of the concave side happenings, and deposition and formation of convex shoals will continue, irrespective of what happens on the concave side. When the concave side is rivetted, or pitched, the river gets deepened as the convex bar continues to progress towards the pitching. Due to the shift of the convex bar, the width between the banks reduces, which increases the velocity, and thus provides a current strong enough to prevent further, extension of convex-bar. In that case, the extension of shoal towards concave side is stopped, but the shoal continues to gain in height.

There are four variables, which govern the meandering process. They are : (*i*) Valley slope, (*ii*) Silt grade and silt charge, (*iii*) Discharge, (*iv*) Bed and side materials and their susceptibility to erosion. All these factors considerably affect the meandering patterns, and all of them are interdependent. On the basis of this theory and experiments, various empirical formulas have been developed for connecting various meander parameters ; *but much remains to be learnt about the meander process.*

Meander parameters. The various meander parameters are shown in Fig. 8.9, and are defined below :

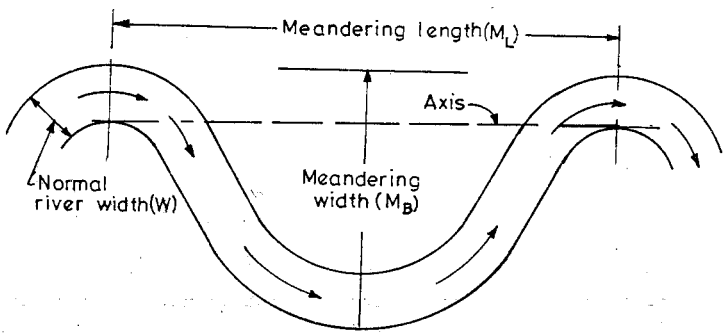


Fig. 8.9. Meander Parameters.

Meander Length (M_L). It is the axial length of one meander, *i.e.* the tangential distance between the corresponding points of a meander.

Meander Belt (M_B). It is the distance between the outer edges of clockwise and anti-clockwise loops of the meander.

Meander Ratio. It is the ratio of meander belt to meander length, *i.e.* $\frac{M_B}{M_L}$

Tortuosity. It is the ratio of the length along the channel (*i.e.* arcual length) to the direct axial length of the river reach.

Crossings or Cross-overs. The short straight reaches of the river, connecting two consecutive clockwise and anti-clockwise loops, are called crossings or cross-overs.

Experiments have been and are being conducted, so as to establish some reliable relationship between these parameters.

A large data on Indian rivers was collected by Jefferson and analysed by Inglis, on the basis of which, he was able to give the following tentative relationship-between different parameters :

Table 8.1 *

Rivers in flood plains (where W is the river width)	*Incised Rivers (Rocky Rivers in hills*)*
$M_L = 6.06\ W$	$M_L = 11.45\ W$
$M_B = 17.40\ W$	$M_B = 27.3\ W$
$M_B = 2.86\ M_L$	$M_B = 2.2\ M_L$
$M_L = 53.6\ Q^{1/2}$	$M_L = 46\ Q^{1/2}$
$M_B = 153.5\ Q^{1/2}$	$M_B = 102.3\ Q^{1/2}$
$W = 8.85\ Q^{1/2}$	$W = 4.53\ Q^{1/2}$

It is generally observed that the total length of a meandering river along the channel remains more or less the same from year to year. If the length is shortened by the development of a cut-off, the original length is regained by rapid erosion and increase in meander belt downstream of cut-off.

It has also been found that the steep rivers meander to a lesser extent compared to those with flatter gradients.

The extent of turtuosity was found to vary throughout the course of the river. How the tortuosity varies, from the point a river becomes alluvial down to its mouth, can be estimated from the meandering of Ganga from Balawalli to the sea, as shown in Table 8.2.

Table 8.2

S.No.	*River reach*	*Direct length in km*	*Length along the channel in km*	*Percentage of tortuosity or percentage of meander*
1.	Balawalli to Garhmukteshwar	104.0	116.8	12
2.	Garhmukteshwar to Rajghat	59.2	67.2	14
3.	Rajghat to Kanpur	280	312	11
4.	Kanpur to Allahabad	176	216	23
5.	Allahabad to Varanasi	136	208	51
6.	Varanasi to Sara	680	864	27
7.	Sara to Sea	296	320	8

For natural rivers, the discharge which determines the meander length and meander belt, may be called the *dominant discharge*. **Dominant discharge** is different from the maximum discharge, because at maximum discharge, the main flow swings away from the bend and erosion at the bank is reduced. Thus, the meander pattern is developed due to the combined effect of the discharge cycle during a flood season, and not by the maximum discharge. In most rivers, the dominant discharge varies between $\frac{1}{2}$ to $\frac{2}{3}$ of the maximum discharge. Meander length for rivers in flood plains may, therefore, be given by

$$M_L = 65.8 \sqrt{Q_{dominant}} \qquad ...(8.1)$$

where $Q_{dominant} = \frac{1}{2}$ to $\frac{2}{3}$ (generally 9/16) of maximum discharge.

Cut off. In an excessively meandering river, a particular bend may sometimes be abandoned by the formation of a straighter and a shorter channel (AB), as shown in Fig. 8.10. The process, whereby, this chord channel is developed or the chord channel itself, is termed as cut-off. It is evident that a meander increases the river length but a cut-off reduces the river length. Hence, a cut-off is a natural phenomenon for counterbalancing the otherwise ever increasing length of the river course due to the development of meanders.

* **Incised river**, as defined in article 8.21(a) represents rocky stage of a river in hills.

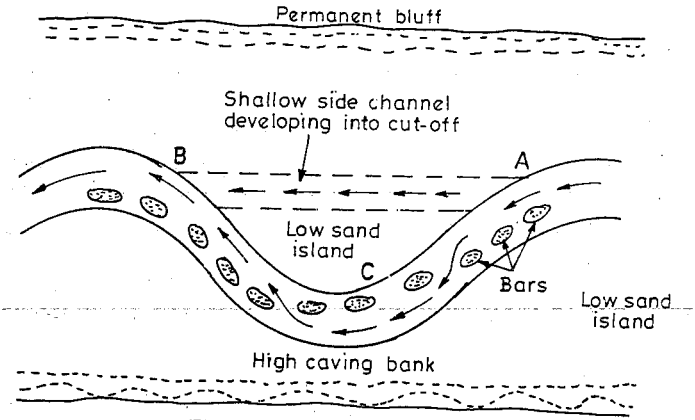


Fig. 8.10. Cut-off development.

Development of a cut-off. A meandering river flowing along the curved path has a shallow side channel besides its main curvilinear path, as shown in Fig. 8.9. This might be the remains of its old course or may be created by floods spilling over the river bank. During high floods, excessive deepening of the pools occur, and is supplemented by the growth of bars at the inflections. Both these factors tend the water to flow more and more towards the side channel. When the flow starts reducing in the main channel, more and more silting occurs in the main channel, which further increases the flow in the side channel. The process continues and finally, a time may come when the entire water starts flowing from the full developed chord channel and the curvilinear path gets silted up.

The period, over which the opening of a chord channel matures into a full-fledged cut-off may vary considerably, depending upon the local conditions. It may take six months, a year, or even five or ten years.

Whenever, a cut-off starts developing, panic and chaos is created over a certain period for miles above and below the newly formed short-cut, as the affected settlements like Jhuggis or agricultural operations, will have to be adjusted. Banks start caving, new channels are formed, old channels may get silted up, until atlast, a temporary stability is created during low floods. The period of adjustment and agitation starts again with the next flood. The process continues, until, full equilibrium conditions are established.

Cut-off ratio. The ratio of the length of the bend to that of the chord, *i.e.* *ACB/AB* (Fig. 8.10) is called the cut-off ratio. This ratio varies depending upon the characteristics of the river at site, such as the discharge, the river flood stage, surface fall, bed material and its suitability for the growth of protective grass and weeds, etc. Hence, only a certain minimum value of cut-off ratio is not sufficient for the development of a cut-off. In other words, a cut-off ratio of 1.7 to 3.0 on a certain reach may result in the formation of a cut-off, while even a cut off ratio equal to 8.0 to 10.0 may not result in the formation of a cut off on some other reach. Normally, cutoff may be developed, if the following conditions are satisfied :

(*i*) The cut-off ratio varies from 1.7 to 3.0 or more.

(*ii*) The ratio $\frac{r}{\sqrt{Q_{max}}}$ is between 13 to 24, where r is the radius of curvature of the loop (bend) ; and Q_{max} is the maximum discharge. A cut-off formation is accelerated if the curvature is too sharp for the discharge.

(*iii*) The shallow side channel is tangential to the main direction of river flow approaching and leaving the cut.

Angle of Swing. The angle of swing is the angle (in degrees) by which the river flow takes a turn (Fig. 8.11). The maximum possible angle of swing was found by Chately, to be equal to

$$\theta = 180° + 2\left[\text{Vers}^{-1}\left(\frac{\text{Chord}}{2 \times \text{Radius}}\right)\right] \quad ...(8.2)$$

This was found on the consideration that when arc/chord ratio exceeds 1.571, *i.e.* when the angle of swing exceeds 180°, conditions favourable for a cut-off are obtained.

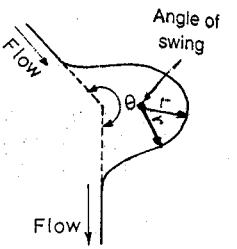


Fig. 8.11. Angle of swing.

8.5. Control and Training of Rivers

River training, in its wider aspects, covers all those engineering works which are constructed on a river, so as to guide and confine the flow to the river channel, and to control and regulate the river bed configuration, thus ensuring safe and effective disposal of floods and sediment loads. Stabilising and training the river along a certain alignment with a suitable waterway is, therefore, the first and formost aim of river training.

8.5.1. Objectives of River Training. The river training works may serve the following objectives or advantages :

(*i*) To prevent the river from changing its course and to avoid outflanking of structures like bridges, weirs, aqueducts, etc.

(*ii*) To prevent flooding of the surrounding countries by providing a safe passage for the flood waters without overtopping the banks.

(*iii*) To protect the river banks by deflecting the river away from the attacked banks.

(*iv*) To ensure effective disposal of sediment load.

(*v*) To provide minimum water depth required for navigation.

8.5.2. Classification of River Training. Depending upon the purpose for which a river training programme is undertaken, the river training works may be classified into the following three categories :

(1) High water training or Training for discharge.

(2) Low water training or Training for depth.

(3) Mean water training or Training for sediment.

(1) **High water training or Training for discharge.** High water training is undertaken with the primary purpose of flood control. It, therefore, aims at providing sufficient river cross-section for the safe passage of maximum flood, and is concerned with making the adjoining area flood-proof, by construction of dykes or levees, etc.

(2) **Low water training or Training for depth.** Low water training is undertaken with the primary purpose of providing sufficient water depth in navigable channels during low water periods. It may be accomplished by concentrating and enhancing the flow in the desired channel by closing other channels, by the process of *bandalling* by contracting the width of the channel with the help of 'groynes', etc.

(3) **Mean water training or Training for sediment.** Mean water training aims at efficient disposal of suspended load and bed load, and thus, to preserve the channel in good shape. The maximum accretion capacity of a river occurs in the vicinity of mean water or dominant discharge. Therefore, the changes in the river bed are attempted in accordance with that stage of flood flow. The mean water training is the most important type and forms the basis on which the former two are planned.

8.5.3. Methods of River Training. The chief aim of river training is to achieve *ultimate stability* of the river with the aid of river-training measures. The *stability* of a river does not mean that changes like scouring and silting of bed, advancement of delta into the sea, etc. will not take place. It only means, that the river attains an equilibrium stage, and no significant change occurs in its alignment, slope, regime, etc. The regime may change within a year but shows little variation from year to year, except that, the river may meander within its *khadirs* (*i.e.* the extreme lines within which the river is ever known to wander).

It was discussed earlier, that a river adjusts its alignment, perimeter, area, slope, etc.; with respect to the discharge and sediment load, either by aggrading, meandering or by degrading. Aggrading and meandering is one and the same thing except that aggrading is the initial stage and meandering is the final stage.

It may be concluded that meandering type is the full and final development of an alluvial river. The other two types are the interim phases and are maintained so long as the factors causing them remain operative. Aggrading rivers are, therefore, not equally amenable to river training on account of their instability. River training works undertaken on aggrading or degrading rivers may, therefore, fail to impose any stability on such rivers. For example, bank protection works undertaken on an aggrading river may either be destroyed by severe erosion or get buried under sediment deposition. *Soil conservation measures in the upper reaches and construction of check dams on tributaries are the most effective measures to be undertaken for controlling aggrading rivers,* before taking up any river training works on such rivers. Similarly, the training works undertaken on degrading rivers may fail due to scour and undermining of foundations by bed scour.

The scouring tendency of degrading rivers must be controlled *by building cross bars, weirs, etc., before attempting any other* river training works on such rivers.

The following are the generally adopted methods for training rivers, including bank protection :

(1) Marginal embankments or Levees.
(2) Guide banks.
(3) Groynes or Spurs.
(4) Artificial cut-offs.
(5) Pitching of banks and provision of launching aprons.
(6) Pitched islands.
(7) Miscellaneous methods, such as Sills, Bandalling, etc.

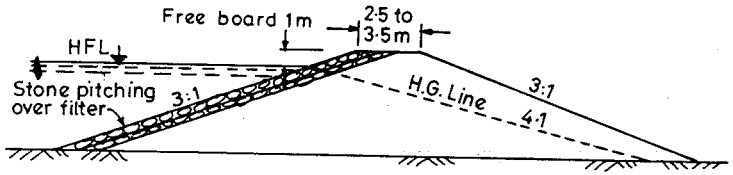
(1) **Marginal Embankments or Levees.** Marginal embankments are generally earthen embankments, running parallel to the river, at some suitable distance from it. They may be constructed on both sides of the river or only one one side, for some suitable river length, where the river is passing through towns or cities or any other places of importance. These embankment-walls, retain the flood water and thus, preventing it from spreading into the nearby lands and towns. *A levee or a dyke is mainly used for flood protection by controlling the river and not by training the river.*

The alignment of levees should follow the normal meandering pattern of the river. The retirement of the levees has to be governed by technical as well as economical and political considerations, because the land falling within the levees, is either to be acquired by the government or remains susceptible to floods. The levees, are many a times, pitched on the upstream side (*i.e.* water side). Launching apron may also be provided, if the bank or levee is close to the main river channel.

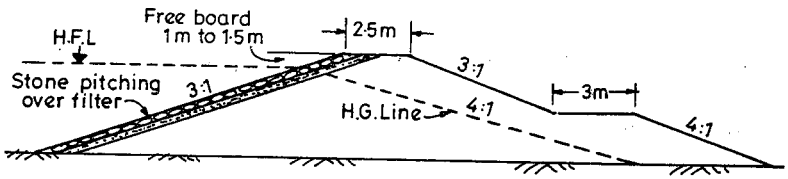
Design of levee section. Levees are just like earthen dams with the difference that they are very long, come in operation discontinuously and for a short time, and have

limited possibilities for selection of their alignment along favourable geological strata. Their sections should be designed in such a way as to keep the seepage gradient inside the body of the embankment by at least one metre from below the top surface of the embankment. The normal value of usually adopted seepage gradient varies between 4 : 1 to 6 : 1 (*i.e.* H : V) depending upon the character of the soil which may necessitate river side slopes varying between 2 : 1 to 5 : 1, land side slopes between 2 : 1 to 7 : 1, and top width between 2.5 to 10 m. The top level of the levee should be decided by leaving a sufficient freeboard varying between 0.3 to 1.5 m above the high flood level.

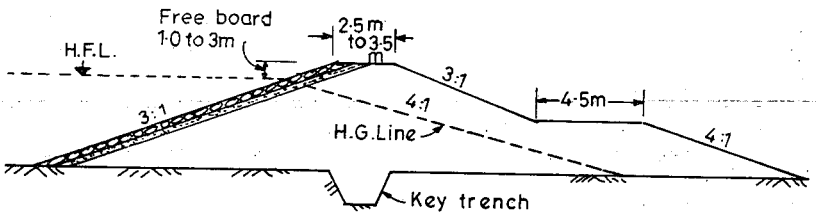
The levee sections which are usually adopted for different heights are shown in Fig. 8.12.



(*a*) For heights below 2.5 m.



(*b*) For heights above 2.5 m and upto 3.25 m



(*c*) For heights above 3.25 m

Fig. 8.12. Typical sections of levees for different heights.

Effects of levees on river bed. In order to assess the effects of dikes or levees on rivers, we shall, first of all, discuss the hydraulics of an undiked river during floods, and then discuss the hydraulics of a diked river.

During floods, water spills over the natural banks of a river. The velocity of this water will be reduced because it spreads over the adjoining land. Due to this reduction in velocity, soil gets deposited as the silt deposition varies inversely with the velocity. In an undiked alluvial river, the flood water spreads over a vast area, so that the velocity becomes so small that almost the entire silt gets deposited on the flood area, and clear water flows back into the main river as the flood recedes.

When the river is enclosed by dikes, the water spills over the natural banks of the river and will spread in the area confined between the river and the dikes. Due to this, the velocity of flow reduces and silt gets deposited. But infact, this reduction in velocity in a diked river is much less than that in an undiked river, because the spread area is less in a diked river. Hence, it can be concluded that the silt deposition will be less in a diked river compared to that in an undiked river.

At the same time, it can also be concluded that in a diked river, the silt will be deposited only in the confined fixed area (*i.e.* area enclosed between dikes and river), and therefore, the bed level of the river as well as that of this enclosed area will increase. Hence, the land enclosed between the dikes will appear to be higher than the adjoining land, and this probably leads to false common man belief that "the dikes cause ill-effect of raising the river-bed".

On the other hand, in an undiked river, the level of the entire land will increase and there will not appear any appreciable raising of the bed. Since the clear-cut raising of the bed is not generally visible on undiked rivers, people get misguided, and they feel that there was no bed rise in undiked rivers, and they become against diking stating "dikes cause the ill-effects of raising the river-bed", although it is not a correct statement.

(2) **Guide Banks.** If an engineering structure, such as, a weir or a barrage or a bridge, etc. is constructed across a river, the river width is reduced and trained in such a fashion, as to ensure not only a safe and expeditious disposal of flood water but also to ensure a permanent reasonable width of the waterway for the river flow. It has already been discussed that the alluvial rivers do shift their courses. Now, if today a structure such as a bridge, is constructed across the existing river width, the other day, the river may shift and there may not be any river below the existing bridge at all, and the river may be found to be flowing away from it, necessitating the construction of another structure. This may lead to the extension of the structure for the entire river length between its *khadirs*.

But, it is unwise and uneconomical to span the entire width of the river and to expose the structure to vagaries of attack and deep scour. Hence, a structure such as a weir, or a barrage, or a bridge, etc. is extended in a smaller width of the river, and river water is trained to flow almost axially through this trough without out-flanking the structure. The river is normally trained for this purpose with the help of a pair of guide banks.

The guide banks are generally provided in pairs, symmetrical in plan and may either be kept parallel or may diverge slightly upstream of the works. Symmetrical and parallel guide banks are usually adopted, unless, the local conditions warrant otherwise.

Before the water enters into the trough formed between these two guide banks, the flow may

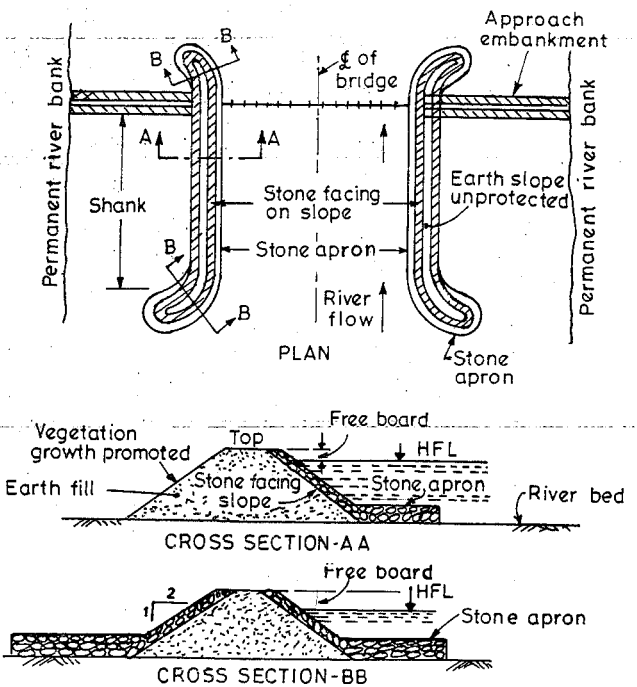


Fig. 8.13. Guide Bank details.

have to be partially controlled and directed with the help of marginal bunds or by groynes or both. The guide banks, usually, consist of two heavily built embankments in the river in the shape of a *bell mouth* (named after the name of its inventer – Mr. Bell). The portion of the river between the normal river banks and the guide banks is closed by ordinary embankments. Sometimes, one of the guide bank may become unnecessary and may be dispensed with, for economical reasons. This may happen either when the khadir bank is very near the works-site, thus serving the purpose of guide bank itself, or when the marginal bund is highly resistant to scouring, general layout of guide banks, plan, section, etc. are shown in Fig. 8.13.

Before we discuss the principles applied for designing the guide banks, it shall be worthwhile to speak something about the selection of the works-site.

Selection of works-site. It is evident that an engineering structure, such as a bridge, or a weir should be spanned in that portion of the river, where the distance between the khadir banks is minimum. This reduces the extent of possible embayment at the back of the guide banks (discussed at little later) and permits shorter guide banks. Further, on a meandering river, the river section at the bend is always wide and non-uniform ; deep on the concave bank and shallow on the convex. While the transition reaches connecting two adjacent bends are narrower and uniform in depth. *Bridges should therefore, preferably be built in these transition reaches, rather than on the bends.*

Further, in case of bridges, the river bed at the proposed bridge site should consist of deep strata of erodible land, so that after constriction, the river may be able to deepen the bed to gain an adequate waterway. If the bed consists of stiff clay etc., the constriction can be done only after due allowance is made for afflux, which may make it constlier.

In case of weirs and barrages, the usual practice is to construct the weir or the barrage, outside the main river channel in a minor creek which is dry in winter, and then to divert the main river channel through it. At the weir site, the river width is constricted. The meandering river upstream has, therefore, to be trained to flow between the two abutments of the weir, without causing any damage.

Principles and Factors Governing the Design of Guide Banks

(*i*) **Top level of guide banks.** The top level of guide banks is governed by HFL, afflux, velocity head, and freeboard. It can be obtained by adding all these four values.

Afflux

By afflux, we mean the rise in the high flood level of the river, upstream of the weir (or the bridge in case of non-erodible soils), or barrage, as a result of its construction. This rise in water level is maximum just near the site of constriction and reduces as we go away from it, upstream. The afflux, extends for a long distance on the upstream side, as shown in Fig. 8.14.

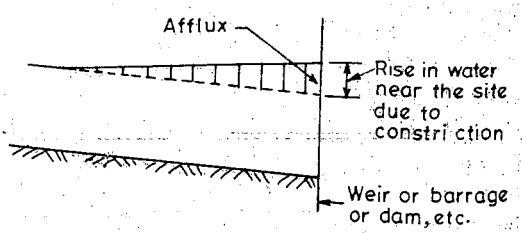


Fig. 8.14. Afflux formation at constriction.

The amount of afflux, in case of weirs or barrages founded on alluvial soils, is generally limited to 1 to 1.2 m, or more commonly as 1.0 m. In steep reaches of the river with boulder or rocky bed and in flashy rivers, a higher value of afflux has to be

taken. The bed erosion and afflux are interlinked. *In case, a bridge is founded on a river, in which full bed scour develops before high floods, a negligible value of afflux may be taken.* While in boulder beds and flashy rivers, the time for bed scour may not be adequate, causing very high afflux. The amount of afflux governs the top levels of the guide banks as well as that of the marginal bunds*. The distance to which the afflux appreciably extends on the upstream side, governs the length and sections of the marginal bunds.

The Waterway and the Discharge per metre run

The waterway is the actual width from which the water has to flow after the river is constricted. Since this width is reduced at the works-site, the depth of water, and hence, the HFL will go up. The rise in this HFL is nothing but afflux. The limit placed on afflux automatically places a limit on the constriction or on the waterway, since the two are interdependent. In addition to high afflux, the insufficient waterway will cause excessive velocities, causing dangerously deep scour along the guide banks at the bridge piers, etc. On the other hand, too long a bridg weir may cause slack velocities, causing formation of shoals with consequent non-uniform flow distribution and an oblique flow.

A likely figure to be adopted for waterway may be given by Lecey's Regime perimeter (P), given by $P = 4.75\sqrt{Q}$; (because in larger rivers, width is approximately equal to its perimeter). In case of a bridge, obstruction caused by piers should be accounted for, and the above equation should be taken to represent the clear effective waterway. It should be remembered that the regime conditions are disturbed after the construction of the weir, and the above formula is not strictly applicable. Most of the existing weirs and bridges have been provided with a clear waterway from 10% to 50% more than that given by Lacey's Regime perimeter.

(*ii*) **Shape of guide banks in plan.** As stated earlier, the guide banks are generally provided in pairs, symmetrical in plan and may either be kept parallel or may diverge slightly upstream of the works. The diverging guide banks may be favoured on the ground that they cover larger width of the khadirs and exert an attracting influence on the flow. But they are not recommended mainly because for equal bank lengths, they provide relatively less protection to the approach embankment, under the worst possible embayment, as shown in Fig. 8.15.

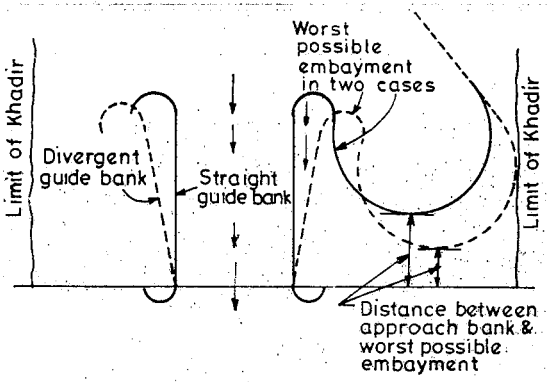


Fig. 8.15. Extent of protection provided by the straight and divergent guide banks.

Moreover, the divergent banks induce oblique flow and tend to form shoals in the centre. However, *in a particular case, where the approaching flow is oblique, it may become necessary to provide divergent banks in order to keep the flow active in adjacent spans.* For example in Yamuna River Bridge in Delhi (opposite old fort), a splay of 10° was given in the left guide bank so as to improve the intensity of flow in the left spans.

* Please see chapter 11 on "Diversion Head Works".

Note. Divergent banks exercise attracting influence on the flow and hence, suitable for oblique flows but bringing the embayment nearer the approach bank and hence, not suitable in ordinary cases.

In a river which is likely to meander on both sides, it is absolutely essential to make the banks symmetrical in order to straighten the current under all possible conditions, and thus, to ensure uniform distribution of discharge and scour. Symmetrical and parallel guide banks are, hence, usually adopted, unless the local conditions warrant otherwise.

(*iii*) **Length of the guide banks**. Spring and Gales have corelated the length of the guide bank with the length of structure between the abutments (L). According to Spring, the length of the upstream part of the guide bank should be equal to 1.1 L or even longer if required to obviate the possibility of the river curving at the back and cutting into the approach bank. The length of the guide bank on the downstream side should be between 0.1 L to 0.2 L.

According to Gales, the upstream length of the guide bank is 1.25 L for flood discharges up to 20,000 cumecs, and 1.5 L for flood discharges more than 20,000 cumecs. The length of the guide bank on the downstream side should be taken equal to 0.25 L. A convergence of 1 in 20 in the former case and 1 in 40 in the letter case has also been suggested by him. His recommendations are based on the assumption that the structure is constructed well within the *khadirs* and is provided with two training bunds. Based on Gales recommendations, the general layout dimensions of guide banks, are shown in Fig. 8.16.

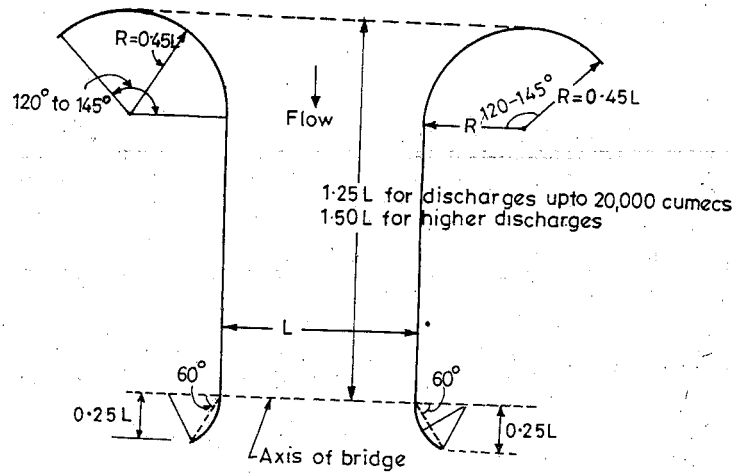


Fig. 8.16. Gale's recommendations for guide banks.

However, in certain cases, where the *khadir* is wide, the Gale's guide bunds may fail to provide enough protection to the approaching embankments. In such cases, the lengths of the guide banks and their layout should be such as to ensure the safety of the approach embankments against the worst possible embayment, in case the river embays considerably behind the training works. Only one loop is likely to be formed if the khadir bank is nearer, as shown in Fig. 8.17 ; and if the distance of the khadir edge is large, a double loop, as shown in Fig. 8.18 ; may be formed. Fig. 8.19 shows the anticipated double loop (dotted) and the actual developed loop on Gomti river u/s of Amghat road bridge.

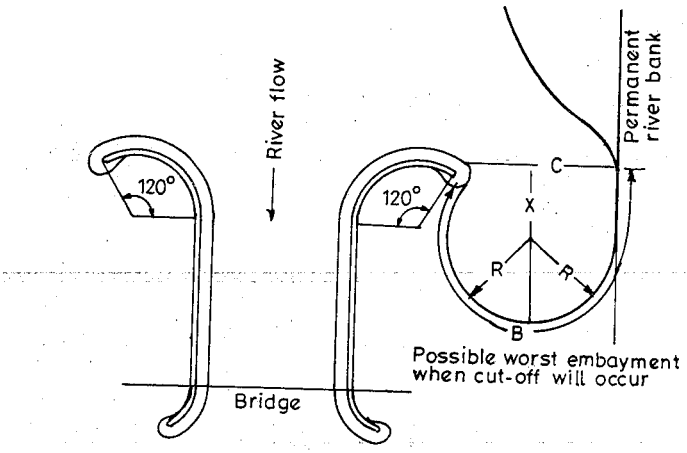


Fig. 8.17. Single loop formation.

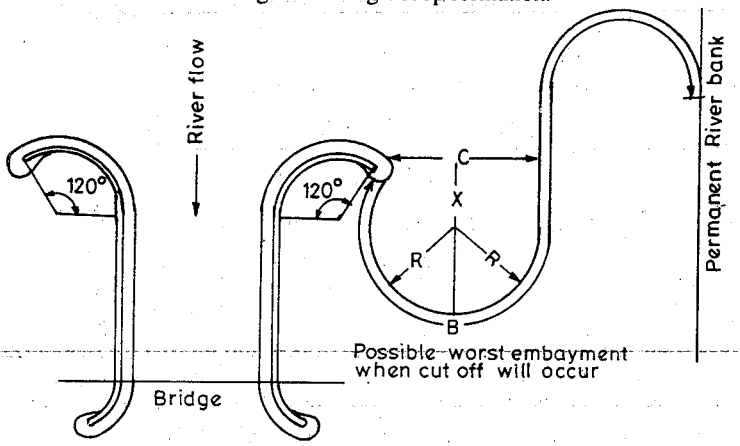


Fig. 8.18. Double loop formation.

Radius of the worst embayment. To determine the radius of the sharpest loop (*i.e.* minimum radius = r_{min}) the river course for the past several years is plotted and average values of M_L, M_B, B and C (where B and C are the arc and chord length) of the river are taken. The average value of the radius of the loop r_{av} may then be determined from the formula

$$\left(\frac{M_L}{4}\right)^2 = \frac{M_B - W}{2}\left[2r_{av} - \frac{M_B - W}{2}\right] \quad \text{...(8.3)}$$

where, M_L = Average meander length in metres

M_B = Average meander belt in metres

W = Average width of the river in the main channel during floods in the vicinity of the structure

r_{av} = Average radius of curvature.

The average value of radius of curvature obtained from the above equation is divided by a suitable constant so as to get the minimum radius of the worst loop. This constant has been found by model experiments to vary between 2.5 to 1.7 for discharges varying from 2000 to 90,000 cumecs, respectively. For acute meanders, B/C ratio is usually in the range of 2.0 to 2.2.

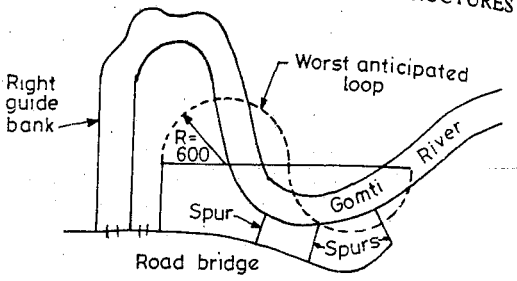


Fig. 8.19. Loops of Gomti River u/s of Amghat Road Bridge.

The length of the guide banks should, therefore, be fixed as per the above requirement and may, therefore, differ from those given by Spring or Gales. The lengths of guide banks should, therefore, be tested and modified by model studies.

(*iv*) **Radius of curved head of the guide banks**

(*a*) *Upstream curved portion.* The upstream curved portion of a guide bank is called 'the impregnable head'. The radius of curvature of the impregnable head should be sufficient enough, as not to cause intense eddies due to the curved flow near it. Greater the radius, flatter the curve, and lesser is the probability of formation of eddies. For a same river slope, coarser the bed material, shorter can be the radius, depending on the expected velocity. A safe value for the radius (R) may be taken equal to

$$R = 0.45\,L.$$

However, Spring suggested a value of R equal to 180 to 250 metres for rivers having velocities between 2.4 to 3.1 m/sec., respectively.

Gales, on the other hand, suggested a value of $R = 250$ metres for rivers having high flood discharge between 7,000 to 20,000 cumecs (with sharper curves permissible for discharges less than 7,000 cumecs); and a value of $R = 580$ metres for discharges varying from 40,000 to 70,000 cumecs. For intermediate discharges *i.e.* between 20,000 to 40,000 cumecs, the value of R may be obtained by interpolation.

The upstream curve is extended to subtend an angle of 120° to 140° at its centre, as shown in Fig. 8.20.

(*b*) *Downstream curved portion.* On the downstream, the river fans out so as to attain its normal width. The downstream portion of the guide bank ensures the safety of approach embankments and prevents the river from attacking them. This purpose can be well served *by providing short guide bund with sharp curved head.* A radius equal to half the radius at the upstream side, may be adopted here and sweep angle of 45° to 60° may be provided, as shown in Fig. 8.20.

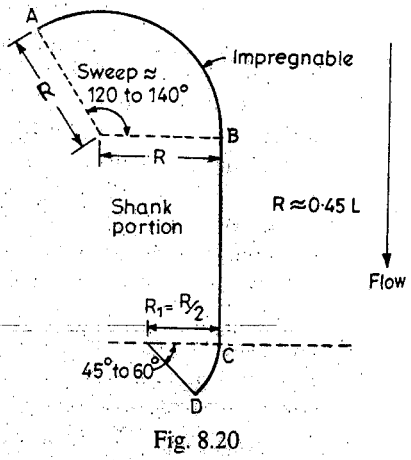


Fig. 8.20

(*v*) **The shank portion and section of bund.** The straight portion of the guide bank *BC* (Fig. 8.19) is called the shank portion. The guide banks should have a minimum top width of 4 metres so as to provide sufficient carriage-way. Extra width may, however,

be provided for storing pitching materials, etc. Side slopes may vary from 1.5 H : 1 V to 2.5 H : 1 V, (but generally kept as 2 H : 1 V), depending on the construction materials and the height of the bund. A freeboard of 1.2 to 1.5 m is generally provided.

(*vi*) **Slope pitching.** The sloping water side of the entire guide bund as well as the sloping rear side of the curved portions are pitched with *one man stone* (*i.e.* a stone which can be lifted by one person—weighing 40 to 50 kg) or concrete blocks. The pitching should extend up to 1 m higher than HFL. The rear side of the shank portion is not pitched, but is generally coated with 0.3 to 0.6 m earth for encouraging vegetation growth, so as to make it resistant against rain, wind, etc. [Refer sections *AA* and *BB* in Figs. 8.22 (*a*) & (*b*)]

The thickness of the pitching on the river side may be calculated by the formula

$$t = 0.06\, Q^{1/3} \qquad ...(8.4)$$

where, t is the thickness of stone pitching in metres
Q is the discharge in cumecs.

The thickness of pitching should be 25% more at the impregnable head than for the rest of the bund.

(*vii*) **Launching apron.** Whenever, a sloping face is protected by stone pitching against scour, the pitching is extended beyond the toe on the bed in the form of packed stones, as shown in Fig. 8.21. This stone dumping is known as *Launching apron.*

If no such protection is provided, scour will occur at the toe with consequent under-mining and collapse of the stone pitching. In order to obviate such a danger to the slope, the pitching is extended on the horizontal river bed portion, which falls down into the scoured portion, as soon as the scour occurs.

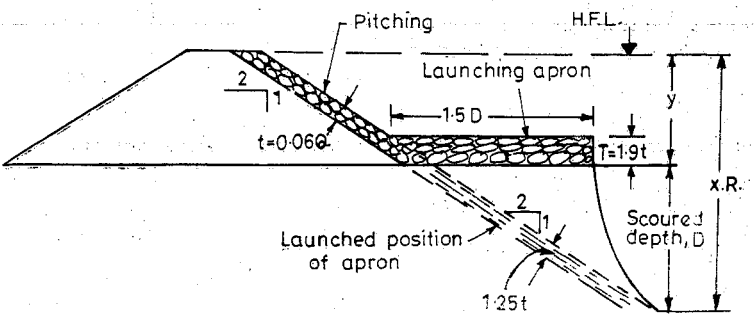


Fig. 8.21. Launching apron details.

The launching apron is generally laid in a width equal to 1.5 times the depth of scour (D) below the original bed. The total scour below HFL is taken as xR, where R is the Lacey's normal scoured depth given by equation

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3} \qquad ...(8.5)$$

where Q is the discharge and f is the silt factor ; and values of x are tabulated for different places, in Table 8.3.

Table 8.3

S. No.	Location	Mean value of x	$D = xR$ – water depth above bed
1	Noses of guide banks	2.25	$2.25R - y$
2	Transitions from noses to straight portions	1.50	$1.5R - y$
3	Straight reaches of guide bunds	1.25	$1.25R - y$

Generally, a scour slope of 2 : 1 is assumed and the thickness of the launched apron should be 1.25 t, where t is the thickness of stone pitching. Then the volume of stone required in the launched apron per unit length perpendicular to paper

$$= \sqrt{2^2 + 1^2} \cdot D \cdot (1.25\,t) = 1.25\,t \times \sqrt{5}\,D = \mathbf{2.8\,t\,.\,D.}$$

If the width of unlaunched apron is 1.5 D, then the thickness of the unlaunched (T) apron is given as :

$$T = \frac{2.8\,t..D}{1.5\,D} = 1.87\,t\,;\ \text{say } 1.9\,t.$$

Hence $\boxed{\mathbf{T = 1.9\,t}}$ ...(8.6)

where t is given by Eq. (8.4).

The width of the apron (1.5 D) will be different in different portions of guide bank, depending upon the values of D as per Table 8.3.

Example 8.1. *The following hydraulic data pertains to a bridge site of a river.*

Maximum discharge = 6,000 cumecs.
Highest flood level = 104.00 m.
River bed level = 100.00 m.
Average diameter of river bed material = 0.10 mm.

Design and sketch Bell's Bunds including the launching apron to train the river. Assume plentiful availability of boulders near the site.

Solution. The waterway between two guide bunds, planned as per Fig. 8.13 may be evaluated as follows :

The Lacey's regime waterway = clear waterway

$$P = 4.75\sqrt{Q} = 4.75\sqrt{6000}\ \text{m} = 368\ \text{m}.$$

Allowing 20% extra for piers, etc., the net spacing between the two guide bunds at the bridge site = $1.2 \times 368 = 440$ m

Hence $L = 440$ m.

∴ The length of the guide bund upstream of bridge (From Fig. 8.16)
$= 1.25 \times 440 = 550$ m.

The length of the guide bund downstream of bridge

$$= 0.25\,L = 0.25 \times 440 = 110\ \text{m}.$$

The radius of the curved head (upstream portion) may be kept

$$= 0.45\,L = 0.45 \times 440 = \text{say } 194\ \text{m}.$$

The u/s end of guide bund may therefore, be curved by 130° (between 120 to 145°) with a radius of 194 m.

The d/s end of the guide bund may, therefore, be curved in such a way as to make an angle of 60°, as shown in Fig. 8.22 (*a*).

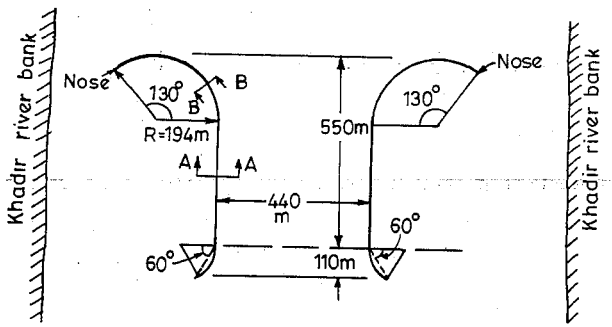


Fig. 8.22 (*a*) Plan of guide bunds for Example 8.1.

Cross-sections of the guide bunds

The given HFL at bridge site = 104.00 m.

Assuming a free-board of 1.5 m, and nil value of afflux, and ignoring velocity head*, we have the

Top level of guide bund = 104.00 + 1.5 = 105.5.

To be more safe and making an allowance for future settlement etc. ; let us adopt the top level of bund as 106.00 m.

Now, height of the bund above river bed level

= 106.00 – 100.00 = 6.00 m.

Assuming the top width of the bund as 5 m, and side slopes as 2 : 1, we have the required sections of the bund as shown in Figs. 8.22 (*b* & *c*).

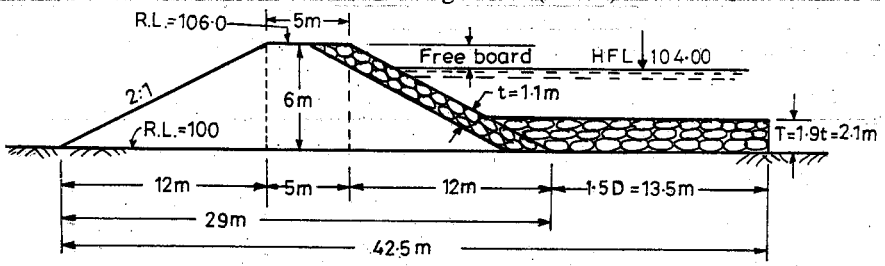


Fig. 8.22 (*b*) Cross-section (*AA*) of the straight reach of Bell's Bund.

The stone pitching and an apron must be provided on the slope on water side for the entire length of the bund ; however, the rear side will also be pitched for curved portions of the bund.

---

*Vel. head $= \frac{V^2}{2g}$

where, $V = \frac{6000}{PR} = \frac{6000}{368 \times 10.36} = 1.57$ m/.sec.

(R is calculated afterwards)

$\therefore$ Vel. head $= \frac{(1.57)^2}{2 \times 9.81} = 0.13$ m.

Design of stone pitching and apron

The thickness of stone pitching on the side (t) is given by Eq. (8.4), as :

$$t = 0.06\, Q^{1/2} = 0.06 \times (6000)^{1/2}$$
$$= 0.06 \times 18.2 = 1.09 \text{ m ; say } 1.1 \text{ m}$$

The thickness of apron is given by eqn. (8.6)

$$= 1.9\, t = 1.9 \times 1.1 = 2.09 \text{ m ; } \textbf{say 2.1 m.}$$

The length of the apron is given by 1.5 D, where D for *the straight reaches* of the guide bund is given as :

$$D = 1.25\, R - y$$

where $$R = 0.47\left(\frac{Q}{f}\right)^{1/3} \qquad ...(8.6)$$

where f is the Lacey/s silt factor given by Eqn. (4.24) as, $f = 1.76\sqrt{d_{mm}}$

where, d_{mm} is the av. particle dia in mm = 0.1 mm (given)

$$f = 1.76\sqrt{0.10} = 1.76 \times 0.322 = \textbf{0.556.}$$

$$\therefore\ R = 0.47\left(\frac{6000}{0.556}\right)^{1/3} = 0.47\,(10780)^{1/3}$$
$$= 0.47 \times 22 = \textbf{10.36 m.}$$

y = Depth of water above river bed level

$$= 104.0 - 100.00 = \textbf{4.00 m.}$$

$$\therefore\quad D = 1.25 \times 10.36 - 4.00 = 13 - 4.0 = \textbf{9.0 m.}$$

Length of apron = $1.5\,D = 1.5 \times 9.0 = 13.5$ m.

For the curvilinear transition portions of the guide bund, however, D is given by

$$D = 1.5\, R - y = 1.5 \times 10.36 - 4.00 = 15.54 - 4.00 = 11.54; \text{ say } 12 \text{ m.}$$

Hence, the length of apron in the curved portions will be $= 1.5 \times 12 \text{ m} = 18$ m. Pitching and apron in these curved portions shall be provided on both sides, as shown in Fig. 8.22 (*c*).

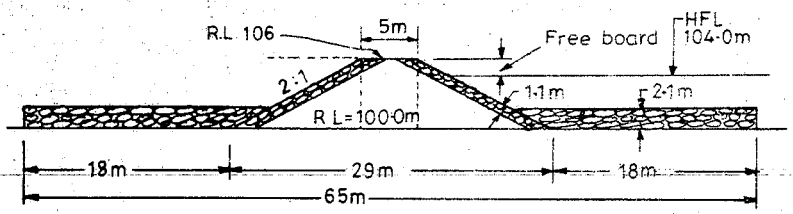
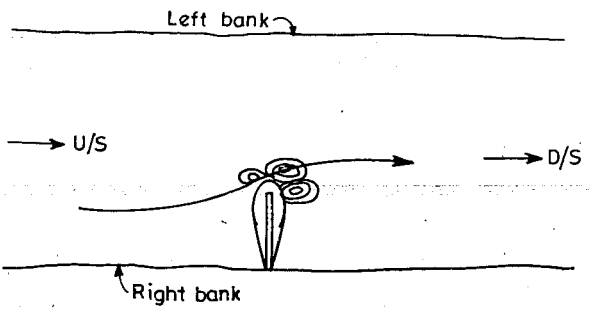


Fig. 8.22 (*c*) Cross-section (*BB*) of the curved portions of the Bell's bund.

The length of apron at noses should be increased to 1.5 D (where $D = 2.24 \times 10.36 - 4.00 = 19.36$) *i.e.* say 30 m.

(3) **Spurs of Groynes.** Groynes are the embankment type structures, constructed transverse to the river flow, extending from the bank into the river. That is why, they

may also be called 'Transverse Dykes'. They are constructed, in order to protect the bank from which they are extended, by deflecting the current away from the bank. As the water is unable to take a sharp embayment, the bank gets protected for certain distance upstream and downstream of the groyne. However, the nose of the groyne is subjected to tremendous action of water, and has to be heavily protected by pitching, etc. The action of eddies reduces from the head towards the bank, and, therefore, the thickness of slope pitching and apron can be reduced accordingly.

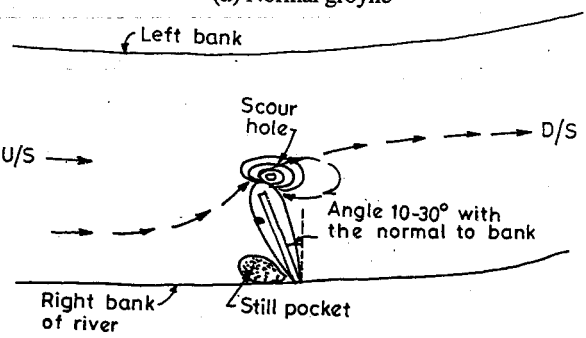


(*a*) Normal groyne

Types of alignment. The groynes may be built either perpendicular to the bank line or they may be inclined upstream or downstream, as shown in Fig. 8.23.

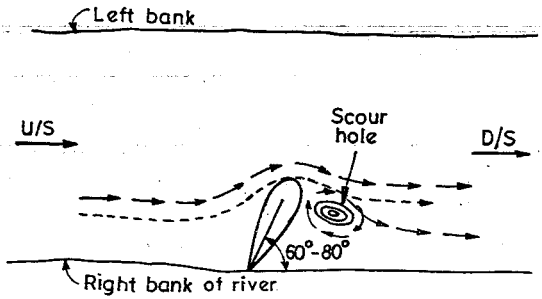
A groyne aligned perpendicular to the bank line [Fig. 8.23(*a*)] is known as an **ordinary groyne** or a **normal groyne.**

A groyne pointing upstream [Fig. 8.23 (*b*)] has the property of *repelling* the flow away from it, and scour holes caused by the formation of vertical eddies are developed away from the bank, and near the head of the groyne. Such groynes, are called **repelling groynes.**



(*b*) Repelling groyne

On the other hand, a groyne pointing downstream [Fig. 8.23 (*c*)], has the property of attracting the flow towards it, and is called an **attracting groyne.**



(*c*) Attracting groyne

Fig. 8.23

In an attracting groyne, scour holes are developed nearer the bank, as compared to those in a repelling groyne. Since such attracting groynes bring the water current as well as scour holes nearer the bank and make it more susceptible to damage, they are generally not used.

The groynes are, therefore, generally aligned either perpendicular to the bank or *pointing upstream*. The 'perpendicular alignment' is generally used on convex banks, and the 'upstream pointing alignment' is generally used on concave banks.

When the length of an upstream pointing groyne is small, such that it changes only the direction of flow, without repelling it, it is called a **deflecting groyne**, instead of calling it a *repelling groyne.*

The repelling groynes are generally found to serve the desired results, provided they are properly located with due regard to their positioning in relation to the meander length. *It is desirable to test their performance in hydraulic models before constructing them in actual field.* These model studies in India are undertaken by Central Water and Power Research Station (CWPRS), Poona. *The angle of deflection upstream is generally kept at about 10 to 30° with a line normal to the bank.*

On the upstream side of a repelling groyne, a still water pocket is formed, where the suspended sediment carried by the river, gets deposited. The head of such a groyne, however, causes severe disturbance in the flow at its nose, where heavy scour takes place. Due to this heavy attack of swirling river current at the head, a repelling groyne needs heavy protection at and near its head, in the form of heavy stone pitching and apron, laid by placing stones filled in wire crates in and around the earthen groyne.

Groynes may be constructed either singly or in series, depending upon the need. When constructed in series, they are more effective, as they create a pool of almost still water between them, which resists the current and gradually accumulates silt between them, thus forming almost a permanent bank after certain time. The choice of using them in series arises, if the reach to be protected is long, or if a single groyne is neither strong enough to deflect the current nor quite effective for silt deposition upstream and downstream of itself.

Length of the groynes. The lengths of the groynes depend upon the position of the existing bank line and the designed or expected bank line for trained river. Too long groynes on easily erodible rivers, are susceptible to damage and failure. In such cases, the best results can be obtained by starting with a shorter length and to extend the groyne gradually, as silting between them proceeds. However, no general rule can be formulated for fixing the length of the groynes. It depends mainly upon the exigencies arising in a specific case. For example, if the entire river course is required to be changed by repelling it towards the opposite bank by means of a single groyne, the groyne must, necessarily be, sufficiently long. Erosion of the opposite bank caused by this shift of water, should be anticipated and allowed for, whenever necessary.

Spacing of the groynes. As each groyne can protect a certain length, the primary factor governing the spacing between two adjacent groynes, is their length. The spacing is, therefore, taken as a certain proportion of their length. Apart from the length, the spacing may be governed by the following factors :

(*a*) *Type of bank where the groyne is to be located.* Larger spacing is required for locating groynes on convex banks, and a smaller one for concave banks with intermediate values at the crossings. A spacing of 2 to 2.5 times the length of the groyne is generally adopted at convex banks, while a spacing equal to the length of the groyne is mostly adopted for concave banks.

(*b*) *The width of the river.* For rivers of equal flood discharges, a larger spacing is preferred for wider rivers than for narrower rivers.

(*c*) *Type of groyne.* A higher value of spacing may be used for permeable groynes as compared to that required for impermeable groynes.

Types of groynes. Based upon their material of construction, the groynes may be divided into two types, namely :

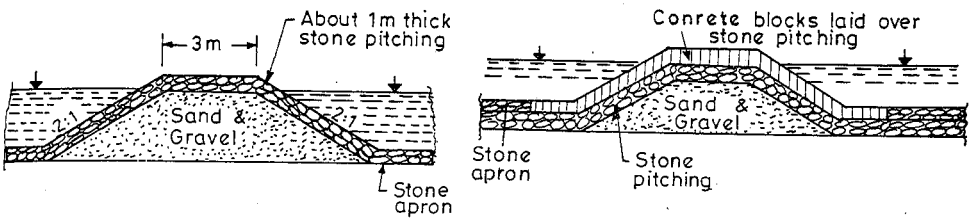
(*i*) *Impermeable groynes* : and

(*ii*) *Permeable groynes.*

(*i*) **Impermeable groynes.** Impermeable groynes are also called as solid groynes or embankment groynes. These groynes may be rockfill embankments or earthern embankments, armoured with stone pitching, concrete blocks, etc. The head of the groyne needs special protection and is generally provided with a launching apron in addition to the increased pitching, as shown in Fig. 8.24 (*b*). These groynes are called 'impermeable groynes' because they do not allow any significant flow through them.

Their design is the same as that for the guide bund with an apron. Commonly adopted dimensions of the groyne embankment are :

Freeboard = 1 m
Top width = 3 m
Side slopes = 2 : 1
Head = square and having a slope of 5 : 1
Apron = as per standard design requirements.



(*a*) Normal section (*b*) Section at the nose (*i.e.* the head)

Fig. 8.24. Impermeable groyne-section.

(*ii*) **Permeable groynes.** Permeable groynes do permit restricted flow through them. The groynes are more or less temporary structures and are susceptible to damage by floating debris, etc. The common materials used as permeable groynes, are :

(*a*) Trees used as groynes, called *Tree Spurs.*
(*b*) Timber stakes or wooden piles called *Balli Spurs.*
(*c*) Stone filled in balli crates.
(*d*) Stone filled in wire crates.

Permeable groynes, simply obstruct the flow, reducing its velocity and causing silt deposition. *They are, therefore, best suited for rivers carrying huge sediment load in suspension.* In comparatively clear rivers, they reduce the erosive strength of the current, and thus, prevent local bank erosion. Permeable spurs do not change the flow abruptly as is done by impermeable spurs, and hence, intense and serious eddies and scour holes are not developed. They are cheaper and perhaps the best for silt laiden rivers. When the groyne is to be submerged, then permeable groynes give much better results, because they do not generate so strong turbulence as is generated by submerged—impermeable groynes, making them susceptible to be washed away due to over-topping.

Balli spurs are being successfully used these days on many rivers in plains, and are becoming very popular.

In such spurs, sal ballis are driven vertically into the river bed, projecting from the bank to be protected, towards the interior of the river, at suitable spacings of about 1 m or so in two rows in zig-zag fashion. The top level of the ballis is kept at about 15 cm above the dry weather flow level of the river. The ballis are connected at top by cross ballis and longitudinally by longitudinal ballis, as shown in Fig. 8.25.

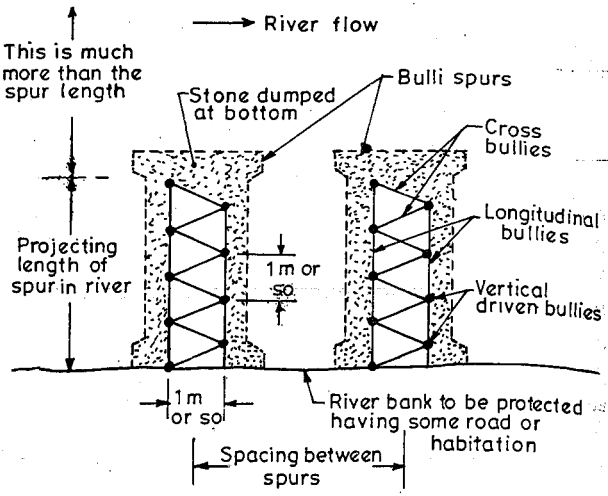


Fig. 8.25. Plan view of Balli spurs constructed in a river.

The length of the spur which projects into the river is kept at about 15 to 20 m or so, and the spacing between the spurs is kept at about 50 m or so. The length and spacings are interdependent on each other. Stone is dumped into the bottom round the spur as shown. The depth of the stone is kept at about 0.8 m or so. The stone so dumped will surround the ballis and prevent scouring and their consequent removal.

The balli spurs so constructed along the river bank, promote silting near this bank; thereby avoiding any erosion of the bank which might have taken place in their absence, endangering the existing roads or establishments in the vicinity of this bank.

Balli spurs constructed on Yamuna river in Delhi at Barapula near Ring road and at Khijrabad village, under the supervision of this author, had proved very successful, leading to enough silting and protecting the bank reach from future erosion. However, their effect was felt downstream, where erosion did take place at Jogabai village.

Although such permeable spurs are excellent tools for training heavily silted rivers, yet their biggest drawback is their short life. They are not strong enough, as to resist shocks and pressure from debris, floating ice and logs of wood, etc. These are, therefore, not suitable for upper reaches of rivers. Moreover during floods, submerged groynes are a danger to navigation.

T-shaped groynes. Denehey's T-shaped groyne is a special type of groyne developed in India. It is an ordinary groyne provided with an extra cross groyne at the

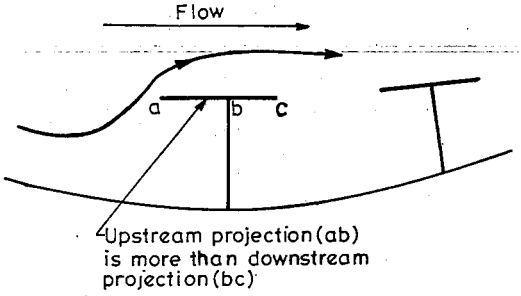


Fig. 8.26. T-shaped groyne

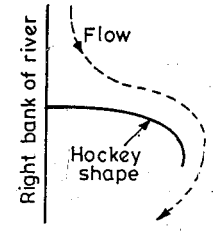


Fig. 8.27. Hockey shaped groyne.

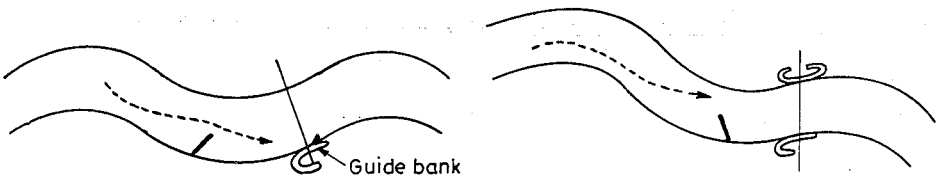
head giving it a T-shape. The cross groyne protects the main groyne on the same principles as the main groyne saves the bank. The longer arm (*ab*) of the T is provided on the upstream, and the shorter one (*bc*), on the downstream, as shown in Fig. 8.26.

These groynes are usually spaced at about 800 metres apart.

Hockey-shaped groynes. These groynes are shaped like a hockey stick at their lower end, as shown in Fig. 8.27.

These groynes exert an 'attracting' type of influence on the flow and hence are not useful for bank protection for repelling the current away from it.

Concluding remarks on groynes. In the end, it may be repeated that the groynes should be constructed after model studies. Their design is not much amenable to theoretical investigation and has to be checked and tested with model studies. Whereas, a series of groynes may be useful for general deflection of the river, a single groyne placed suitably may best serve the purpose of controlling a river at a certain works-site, such as shown in Fig. 8.28. (*a*) and (*b*).



Water attracted towards guide bank, protecting the free bank
(*a*) attracting groyne

Water deflected away from concave side reducing meander tendencies
(*b*) deflecting groyne

Fig. 8.28

(4) **Artificial cut-offs.** It was discussed earlier, that a considerably meandering river may attain its original straight flow by means of a cut-off. A cut-off channel may develop by itself or may be induced artificially. When a meander goes on increasing and may endanger some valuable land or property, then the river course may be straightened by inducing an artificial cut-off. The newly developed course will be far away from the establishments which otherwise would have been affected. For inducing an artificial cut-off, only a pilot channel is required to be excavated in case of rivers having easily erodible beds. The flood water will gradually enlarge **the pilot cut** to the required cross-section and will abandon the old curved channel, provided, the circumstances for the development of this cut-off are favourable.

It has been established that the side pilot channel will be self scouring, provided $\left(\frac{R}{L^2}\right)$ *is greater for the cut than for the original course* (where R is the hydraulic mean depth $\left(R = \frac{A}{P}\right)$ and L is the length along the cut or along the river course as the case may be. Hence, a deeper and more hydraulically efficient channel $\left(\text{such that } \frac{R}{L^2} \text{ is greater for}\right.$ the cut than for the main course) is required to be excavated for inducing a cut-off. *The pilot cut should be made as deep as possible.* As tractive force is directly proportional to depth, a deeper cut would be helpful for rapid development. The width of the cut is relatively less important and is governed by the minimum working space required for the excavating machines, etc.

Moreover, the alignment of the cut should be tangential to the main direction of flow approaching and leaving *the cut.*

It was discussed earlier that a cut-off brings violent changes in the river regime. To keep this violence within permissible limits, a full scale diversion of the river through an artificial cut-off is seldom accomplished at any one time. The complete formation of a cut-off is generally brought about slowly and slowly. The common practice is to make pilot cuts which carry 8 to 10 per cent of discharge in the beginning, and are developed subsequently to carry about 40 to 50 per cent of the total river discharge.

Since a cut-off helps in reducing flood heights and flood periods, it helps in shortening the navigation course and also helps in protecting endangered establishments. It is, thus, a useful river training measure. However, cut-offs as river training measure are not enough by themselves. Although they correct the instability and inefficiency at sharp bends, they do little to correct conditions in the reaches between these bends. For this reason, extensive supplementary training works are required between the cut-offs. The main purpose of these supplementary training works is the creation of a uniform river width and establishment of a central river channel deep enough to maintain itself by normal scour action. Construction of groynes and revetments at places, which are susceptible to erosion, are the common supplementary training works.

(5) **Pitching of banks and provision of launching aprons**. Protection of banks is a part and parcel of river training works. Sometimes 'Bank protection' is separated from the 'River Training' under the argument that the former has a limited function of protecting the bank, while the latter aims at training the river to a desired stable course. However, since bank protection itself helps in training the river, the two are generally treated as one and the same thing.

Banks of a river are directly protected by stone pitching, or by concrete blocks, or by brick lining or by growing vegetative cover, etc. Concrete blocks are very costly, and stone pitching is mostly adopted, if available without much difficulty. The banks of the river are made stable by giving them a stable slope varying from 1 : 1 to 2 : 1 depending upon the material of the bank. They are then pitched, so as to make them strong enough to resist erosion.

Sometimes, as discussed earlier, a launching apron is projected from the toe of the bank into the river, so as to prevent scour at the toe and the consequent fall of slope pitching. The standard design of an apron has already been discussed and is based on the principle that the stone in the apron has to launch into the deepest scour (D) possible at the location, to a slope of 2 : 1, at an average thickness of 1.25 times the thickness of pitching provided at the bank. The deepest scour may be taken equal to x times the Lacey's Regime scoured depth ; x generally varying from 1 to 2.5 or 3 for different places.

(6) **Pitched islands.** A 'pitched island' is an artificially constructed island in the river bed and is protected by stone pitching on all sides. Because of the turbulence generated by the island in its vicinity, the river channel around the island gets deepened and thus, attracting the river towards itself and holding it permanently. Pitched islands may therefore, help in attracting the current towards themselves and thus, reduce undue concentration on the opposite banks.

The device of pitched island as a river training measure is of a recent origin and is still in the experimental stages. Nevertheless, *experience* has proved that pitched island is a very effective means of controlling and training rivers, especially in the vicinity of

control points, such as a bridge or a weir or a barrage, etc. However, model studies must be conducted before adopting pitched island, as a river training measure.

(7) **Miscellaneous methods.** (*i*) *Submerged Dikes* (called Sills) are used as a river training measure in particular situations. Sometimes, a river may create deep channels in the vicinity of certain pucca structures and are required to be corrected. In such situations, **sills** are placed across the scoured portion of the bed, with their top levels at or slightly below the designed bed level aspired to be achieved after correcting the deep scours. They are spaced closely, so as to ensure their proper functioning.

(*ii*) *Closing Dykes* are sometimes used to close a particular flow, so that the river flow may be directed in some other desired direction.

Conclusions and remarks. River training works depend mainly upon the type of the river, its regime and upon the characteristics of its flow. *There are no two countries with the same hydrology ; in the same country, there are no two rivers with the same regime ; and even in the same river, there are no two floods of the same characteristics. River training works, therefore, vary from place to place and from time to time and also depend upon the finances available at a particular time.* A particular method of river training, which has been successful in a particular reach on a particular river, may not be the best solution for the other rivers or for other reaches of the same river.

Hence, no particular method of river control can be stated to be the best. It all depends upon the exigencies arising out of a particular problem. However, in general, it can be stated that no river training method is of immense use in isolation, unless it is supplemented by other measures. A comprehensive planning and model testing is necessary before any measures are undertaken to control and train the river. If roughly planned, costly structures may either be washed away in a few days or may create worse or irreversible consequences causing tremendous losses and devastations.

PROBLEMS

1. Comment on the statement—"Rivers have played an important part in the development of civilisation right from the early ages."

2. (*a*) What is meant by "meandering of rivers" and what are its causes ? How does the increasing river length due to meandering is counteracted in nature ?

(*b*) Write short notes on the following river training works :

(*i*) Levees (*ii*) Guide banks (*iii*) Repelling groynes (*iv*) Cut-off

3. Give a typical plan and section of Bell's bund, and explain its design procedure step by step.

Differentiate between 'attracting' and 'repelling' spurs explaining their action in river training.

4. What are 'cut-offs' ? How are they used as a method of river training ?

Draw a neat plan and section of a Bells Bund and give salient steps of its design procedure.

5. (*a*) What are the different types of rivers and their characteristics ?

(*b*) Name the important Himalayan Indian rivers, and compare their behaviours with the other Non-Himalyan Indian rivers.

(*c*) Name the important types of river training methods indicating the purpose for which each type is adopted.

6. Explain how do the following assist in river control :

(*i*) Spurs (*ii*) Revetment (*iii*) Guide bunds.

Draw a neat plan and section of an earthen Tee-spur showing protective apron.

State the good and evil effects of embankment construction for flood control.

7. What is the difference between permeable and impermeable spurs ? Describe briefly the type of spurs commonly used in river control.

Data given below refer to a Bell's bund :

High flood level	206.00 m.
Low water bed level	202.00 m.
Maximum scour depth (Lacey's formula) R (with factor or safety)	9.5 m.
Free-board	1.5 m.
Side slope of embankment	2 : 1
Thickness of stone apron when launched	40 cm.

Determine volume of stone required per metre length of shank. (Assume 25% loss). Draw a neat sectional view of the bund with necessary dimensions.

8. (*a*) What is meant by 'river training' and what are the different objectives served by it.

(*b*) Enumerate the different methods which are used for controlling and training rivers, and describe any one of these methods in details. Also discuss the circumstances under which a particular method of river training is to be adopted.

9. (*a*) Explain with neat sketches the different types of 'spurs' which are commonly used for controlling and training Indian rivers ?

(*b*) Discuss briefly the usefulness of the 'Balli spurs' on North Indian rivers.

(*c*) What are 'cut-offs' and how are they artificially induced ? What are their advantages and disadvantages ?

10. Write short notes on any three of the following :

(*i*) Importance of rivers and necessity of controlling them.

(*ii*) Aggrading and degrading rivers.

(*iii*) Meandering of rivers and meander parameters.

(*iv*) Cut-offs and cut-offs ratio.

(*v*) High water, low water, and mean water training.

(*vi*) Pitching of Bank's and Pitched island's as the means of training rivers.

(*vii*) Groynes, their types and uses.

(*viii*) Use of Levees for protecting cities from floods.

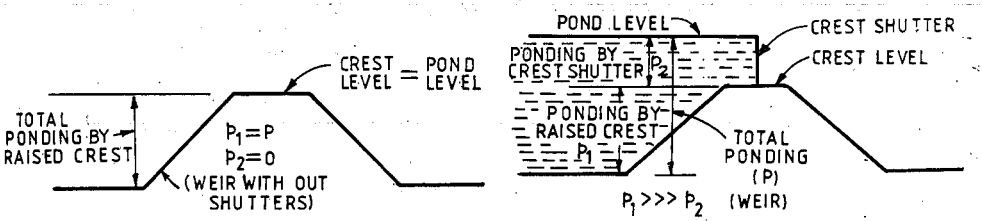
9

Diversion Head Works

It was stated earlier that the main permanent canal, forming the primary part of a direct irrigation scheme, takes off from a diversion weir or a barrage. In fact, these permanent canals take off from rivers and the arrangements are so well made at their heads, that a constant and a continuous water supply is ensured into the canal, even during the periods of low flow. *The works, which are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal, are known as Diversion Head Works.*

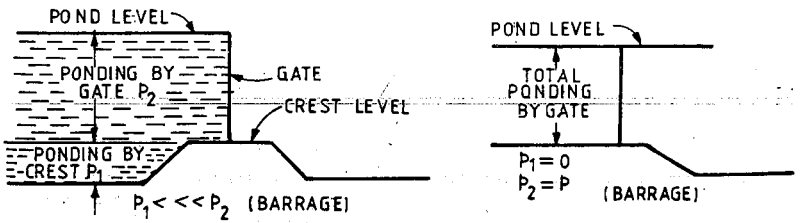
9.1. Weir and Barrage

In general, the above purpose can be accomplished by constructing a barrier across the river, so as to raise the water level on the upstream side of the obstruction, and thus, to feed the main canals taking off from its upstream side at one or both of its flanks. The ponding of water can be achieved either only by a permanent pucca raised crest across the river or by a raised crest supplemented by falling counter-balanced gates or shutters, working over the crest. *If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is achieved by the shutters, then this barrier is known as a weir [Fig. 9.1 (a) and (b)]. On the other hand, if most of the ponding is done by gates and a smaller or nil part of it is done by the raised crest, then the barrier is known as a Barrage or a River Regulator [See Fig. 9.2 (a) and (b)].*



(*a*) Weir without shutters (*b*) Weir with shutters

Fig. 9.1



(*a*) Barrage with a small raised crest (*b*) Barrage without any raised crest

Fig. 9.2

If most of the ponding or the entire ponding is done by a permanent raised crest, as in a weir, then the afflux caused during high floods is quite high. On the other hand, if most of the ponding is done by gates, as in a barrage, then the gates can be opened during high floods and the afflux (*i.e.* rise in HFL near the site) will be nil or minimum. Hence, the latter device *i.e.* a barrage, gives less afflux and a better control upon the river flow, because the inflow and outflow can be controlled to a much greater extent by suitable manipulations of its gates.

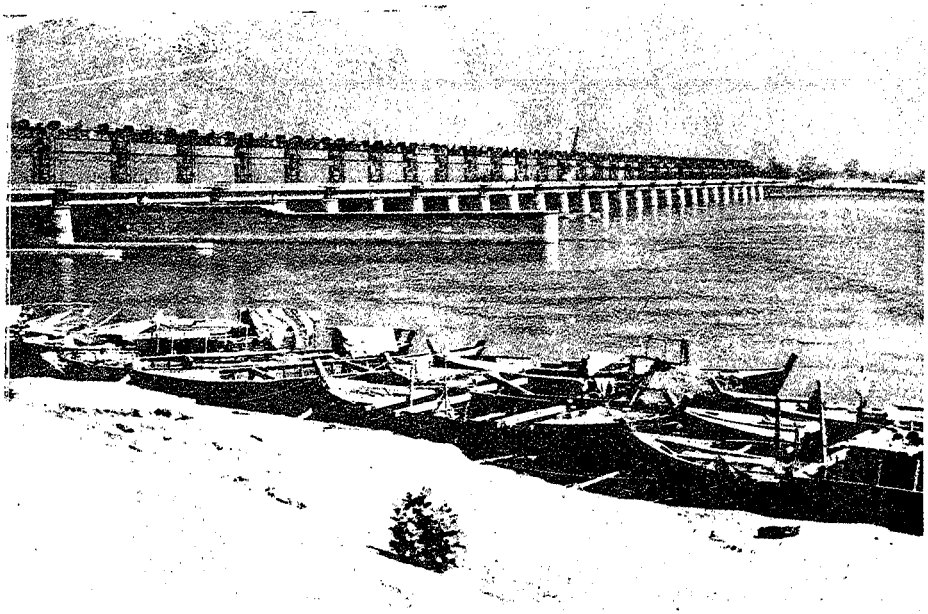


Fig. 9.3. Photoview of a Barrage (Girija Barrage on Ghagra river in U.P.)

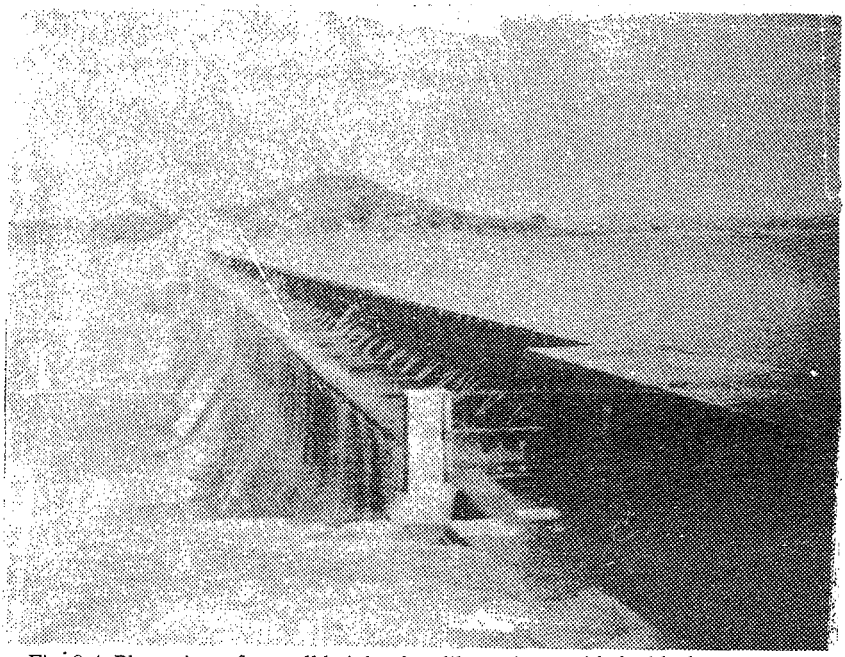


Fig. 9.4. Photoview of a small height. dam like weir, provided with shutters at top of its solid crest (Devri Dam near Jhansi in U.P.)

Moreover, a gate controlled weir (*i.e.* a barrage) can be provided with a roadway across the river at a small additional cost. The choice between a weir with shutters and one with counter-balanced gates (*i.e.* a barrage) is largely a matter of cost and convenience in working. A shuttered weir will be relatively cheaper but will lack the effective control possible in the case of a barrage. Moreover, a barrage type construction can be easily supplemented with a roadway across the river at a small additional cost. *Hence, barrages are almost invariably constructed these days on all important rivers.*

9.2. Gravity and Non-Gravity Weirs

When the weight of the weir (*i.e.* its body and floor) balances the uplift pressure caused by the head of the water seeping below the weir, it is called a *Gravity weir.* On the other hand, if the weir floor is designed continuous with the divide piers as reinforced structure, such that the weight of concrete slab together with the weight of divide piers, keep the structure safe against the uplift ; then the structure may be called as a *Non-gravity Weir.* In the latter case, RCC has to be used in place of brick piers, but in particular cases, considerable savings may be obtained, as the weight of the floor can be much less than what is required in a gravity weir.

9.3. Layout of a Diversion Head Works and its Components

A typical layout of a canal head-works is shown in Fig. 9.5. Such a head-works consists of :

(1) *Weir proper.*

(2) *Under-sluices.*

(3) *Divide wall,* dividing the river width into two portions ; one is called the weir portion, and the other portion from which the canal takes off, is having openings* and called the 'under-sluice-pocket' or *'under-sluices'* or *'weir scouring sluices'*. If there are two canals, taking off from each flank, then there will be two divide walls and two under-sluices.

(4) *River training works,* such as marginal bunds, guide banks, groynes, etc.

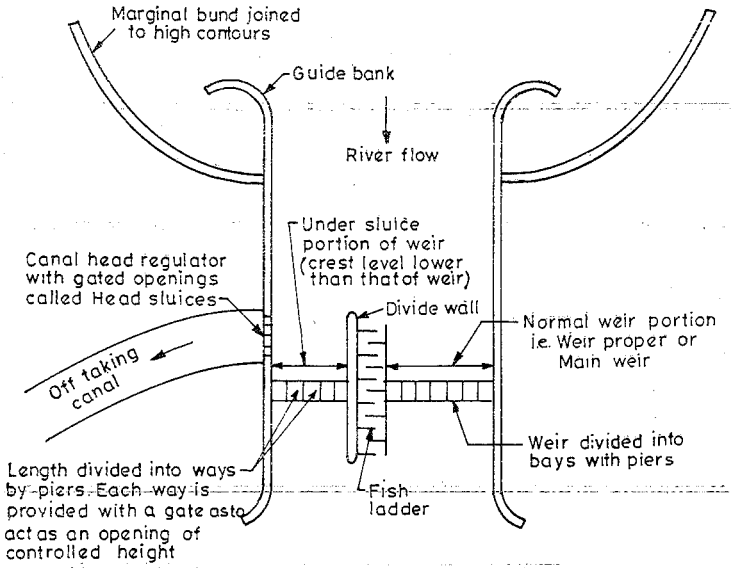


Fig. 9.5. Typical Layout of Diversion Head-Works.

* These openings are made by partial lifting of gates of the under-sluice ways. During high-floods the gates of the under-sluice ways shall be fully lifted depending on excess discharge in the river.

(5) *Fish Ladder.*
(6) *Canal Head Regulator.*
(7) *Weir's ancillary works,* such as shutters, gates, etc.
(8) *Silt Regulation Works.*

The various parts of a diversion head-works scheme and their design considerations are explained in details in the following pages.

9.3.1. The Diversion Weir and its Types. *Alignment.* As stated earlier, *a diversion weir or an anycut or an intake weir* is a raised pucca structure with or without shutters and laid across the river width. It is, essentially, of a height say up to 9 metres or so. The height of shutters over the weir crest seldom exceeds 1.2 metres or so. The entire length of the weir is divided into a number of bays by means of divide piers so as to avoid cross-flow in floods. As far as possible, the weirs should be aligned at right angle to the direction of the main river current. This ensures lesser length of the weir, better discharging capacity and lesser cost. This right-angled alignment is better and, therefore, common, especially when the river bed is silty or sandy. Sometimes, the weir may be aligned at an oblique angle to the direction of the river current, and thereby, obtaining more safe and better foundations. In such a case, the weir will be of greater length, will have less discharging power and will be costlier. Moreover, due to non-axial flow, cross-currents may be developed, which may undermine the weir foundation. An oblique alignment may sometimes become necessary when the river bed consists of gravel and shingle, which could otherwise enter the head regulator of the main canal and get deposited into the head reach of the main canal.

Types of Weirs. The weirs may be divided into the following three classes :

(*i*) *Masonry weirs with vertical drop ;*
(*ii*) *Rock-fill weirs with sloping aprons* ; and
(*iii*) *Concrete weirs with sloping glacis.*

These three important types of weirs are described below :

(*i*) **Masonry weirs with vertical drop.** A typical cross-section of such a weir is shown in Fig. 9.6.

The above type of a weir, consists of a horizontal floor and a masonry crest with vertical or nearly vertical downstream face.

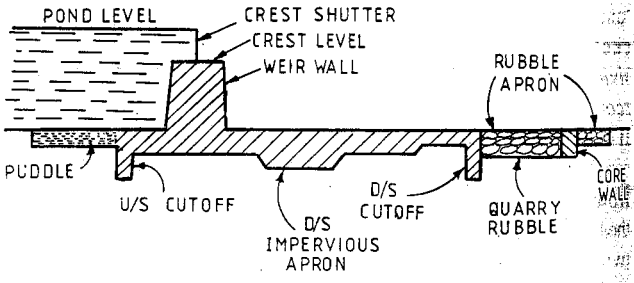


Fig. 9.6. Masonry Weir.

The raised masonry crest does the maximum ponding of water, but a part of it, is usually, done by shutters at the top of the crest. The shutters can be dropped down during floods, so as to reduce the afflux by increasing the waterway opening.

This type of weir was used in all the old head-works, such as Bhimgoda, Rasul, Khanki and Marala works, etc. and is particularly suitable for hard clay and consolidated gravel foundations. However, this type of weir is becoming obsolete and even the old constructions are being replaced by the new modern concrete weirs.

(*ii*) **Rock-fill weirs with sloping aprons.** Such a weir is also called — '*Dry Stone Slope Weir*'. A typical cross-section of such a weir is shown in Fig. 9.7. It is the simplest

type of construction, and is suitable for fine sandy foundations like those in alluvial areas in North India.

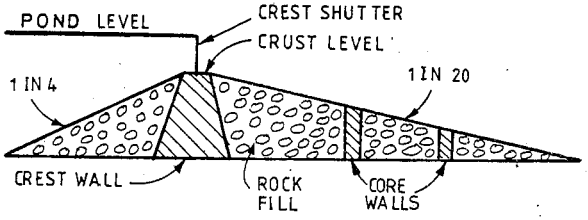


Fig. 9.7. Rock-fill Weir.

The old Okhla weir* across the Yamuna river in Delhi provides an example of such a weir. Such a weir requires huge quantities of stone and is economical only when stone is easily available. The stability of such a weir is not amenable to theoretical treatment.

However, with the development of concrete glacis weirs, the above type is also becoming obsolete.

(iii) **Modern concrete weirs with sloping downstream glacis.** Weirs of this type are of recent origin and their design is based on modern concepts of sub-surface flow (*i.e.* Khosla's Theory). A typical cross-section of such a weir is shown in Fig. 9.8. Sheet piles of sufficient depths are driven at the ends of upstream and downstream floor. Sometimes, an intermediate pile line is also provided. *The hydraulic jump is formed on the downstream sloping glacis, so as to dissipate the energy of the flowing water.*

This type of weirs are now exclusively used, especially, on permeable foundations, and are generally provided with a low crest with counterbalanced gates, thus, making it a barrage. A typical cross-section of a barrage is shown in Fig. 9.9.

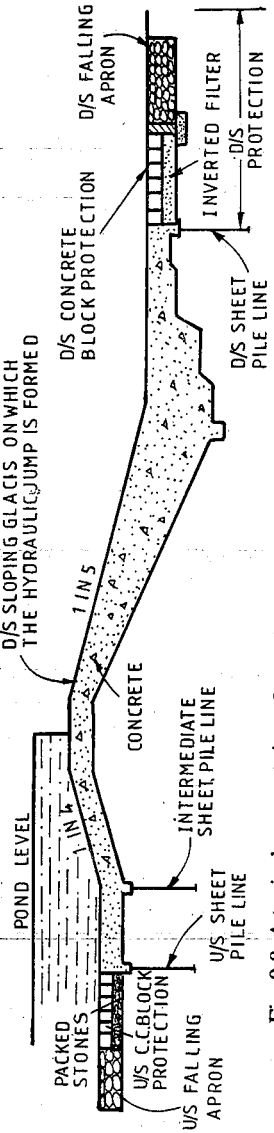


Fig. 9.8. A typical cross-section of a modern concrete weir founded on permeable foundations.

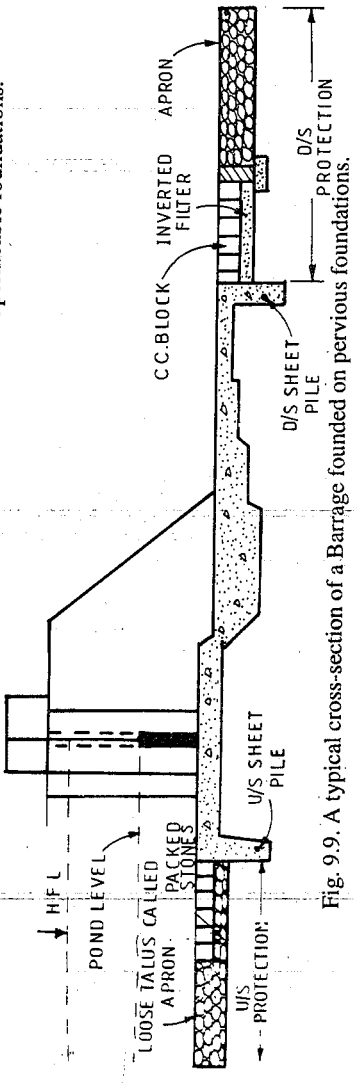


Fig. 9.9. A typical cross-section of a Barrage founded on pervious foundations.

* Now replaced by a new barrage, about 2 km downstream. The old weir has however, not been dismantled.

The barrage shown here is not having a raised crest at all, but barrages can do have slightly raised crests. The detailed design of such a weir or a barrage will be discussed in Chapter 11.

9.3.2. Afflux and Pond Level. The rise in the maximum flood level (HFL) upstream of the weir, caused due to the construction of the weir across the river, is called **afflux.**

In the initial stages, the water level is raised on account of afflux up to small distance equal to the length of the back water curve. But with the passage of time, the river bed rises due to silting caused by the reduction in flow-velocity upstream of the weir. (Q remaining same, y or A increasing and V decreasing). The effect, therefore, travels upstream till the river bed slope upstream of the weir is the same as was before its construction. (See Fig. 11.18 in Chapter 11).

Pond Level. The water-level required in the under-sluice pocket upstream of the canal head regulator, so as to feed the canal with its full supply, is known as *Pond Level.* The FSL of the canal at the head, depends upon the level of the irrigated areas and the slope of the canal. The pond level is generally obtained by adding 1.0 to 1.2 m to canal full supply level.

Since the weir top is raised upto the pond level, a minimum water level equal to the pond level is always maintained in the under-sluice pocket, so as to ensure a continuous supply of water into the canal with its full supply level.

The available head at the canal head regulator is, therefore, equal to the difference of the pond level and canal FSL. The width of regulator openings (*i.e.* waterway) should, therefore, be governed by this available head. But to make provision for future silting of canal, and for the subsequent rise in FSL, (which correspondingly reduces the available head), the designed available head for its water-way is taken at about $\frac{1}{2}$ of the original available value.

This will be explained in details in the design of Canal Head Regulators (in Chapter 11).

9.3.3. The Under-Sluices or Scouring Sluices. A comparatively less turbulent pocket of water is created near the canal head regulator by constructing under-sluice portion of the weir. A divide wall separates the main weir portion from the under-sluice portion of the weir. *The crest of the under-sluice portion of the weir is kept at a lower level than the crest of the normal portion of the weir.*

Normally, the crest level of the under-sluices is kept equal to the deepest bed level of the river during non-monsoon season; whereas, the crest level of the 'weir' is kept higher by about 1 to 1.5 m.

As the crest of the under-sluice pocket is at a low level, a deep channel develops towards this pocket, which helps in bringing low dry-weather discharge towards this pocket, thereby, ensuring easy diversion of water into the canal through the canal head regulator.

The under-sluiced length of weir or barrage is divided into a number of ways by piers, and separate gates are installed on these ways. Each way can thus, be opened to any desired height by lifting its gate. Each way can thus, act as a gate controlled opening, and will help in bypassing the excess supplies to the down-stream side of the river. These openings will also help in scouring and removing the deposited silt from the under-sluiced pocket ; and hence are also called the *scouring sluices.*

The crest level of the head regulator is also kept higher than the crest level of the under-sluices, so that only silt free water is admitted into the canal through the *head sluices*. Silt gets deposited over the under-sluice floor, and may be periodically removed over the crest of the under-sluices and towards the downstream side of the river by opening these gate-controlled openings (under-sluices). Since the under-sluices help in removing the silt from near the head regulator, they are also called *scouring sluices.*

The spans of under-sluices (*i.e.* span between two adjoining piers) should be wide enough, to be efficient in scouring action. Stoney gates which can be easily operated from platform at top, are widely used for controlling the flow through the under-sluices. In closed position, the top of these gates should be above the pond level in the pocket, so as to ensure a continuous and a constant supply of water into the off-taking canal through the canal head regulator.

Apart from these two primary objectives served by the under-sluices in all types of weirs and barrages, they may serve a third function in the case of weirs provided with shutters. That is : they help in passing the dry weather flow and low floods without dropping the weir shutters, the raising of which entails a good deal of labour and time. Whenever, water is surplus to canal requirement, it has to be passed across the weir. The surplus water is passed through the under-sluices, so long as it does not exceed their discharging capacity, and the weir shutters are kept raised up. The weir shutters are dropped only to pass down the high floods.

Design considerations. Sill of the under-sluice pocket is kept at or slightly above the deepest river bed and about 0.9 to 1.8 metres below the sill of the canal head regulator. The length of the under-sluice pocket between the divide wall and the head regulator may be taken as 1.5 times the upstream length of the divide wall, as shown in Fig. 9.10.

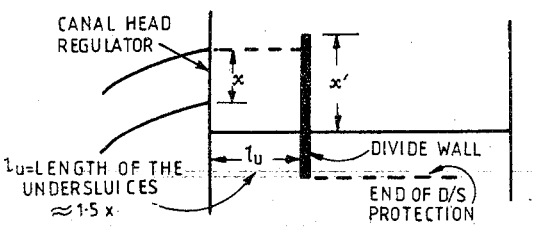


Fig. 9.10

However, this length is governed by the discharging capacity of the under-sluices, which should be sufficient to enable them to serve their main functions, described earlier. The discharging capacity of under-sluices may be selected as follows :

(*i*) They should be able to ensure sufficient scouring capacity, for which the discharging capacity should be atleast twice the full supply discharge of the main canal at its head.

(*ii*) They should be able to pass the dry weather-flow and low floods during the months excluding the rainy season, without the necessity of dropping the weir shutters.

(*iii*) They should be able to dispose of 10 to 15% of the high flood discharge during severe floods.

The spans of the under-sluices should be wide enough (usually 10 to 20 m) in order to be efficient in scouring action. The section of the under-sluices will be similar to that of a weir and will be designed to ensure–proper hydraulic jump formation (from surface flow considerations) and for uplift pressure for pervious foundations (by Khosla's Theory) as per procedure adopted for a normal weir design. Discharge intensity being

greater in the under-sluices, its floor and apron will be stronger as compared to those of weir proper. The floor downstream of the crest, should not be kept level with the crest level, because the formation of hydraulic jump as destroyer of energy is very uncertain on a level floor.

9.3.4. The Divide Wall. The 'divide wall' is a masonry or a concrete wall constructed at right angle to the axis of the weir, and separates the 'weir proper' from the 'under-sluices'. The divide wall extends on the upstream side beyond the beginning of the canal head regulator; and on the downstream side, it extends up to the end of loose protection of the under-sluices (*see* Fig. 9.10). The top width of divide wall is about 1.5 to 2.5 metres. These walls are founded on wells closely spaced beyond the pucca floor upto the end. The wells are taken well below the deepest possible scour. Typical cross-section of the divide wall on pucca floor and beyond the pucca floor are shown in Fig. 9.11 (*a*) and (*b*).

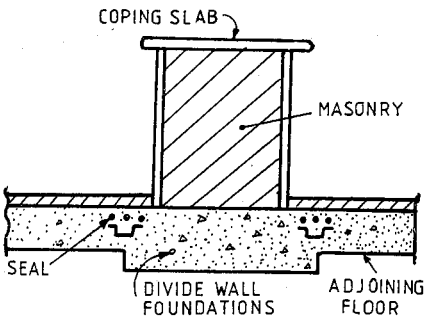


Fig. 9.11. (*a*) Cross-section of Divide Wall on Pucca floor.

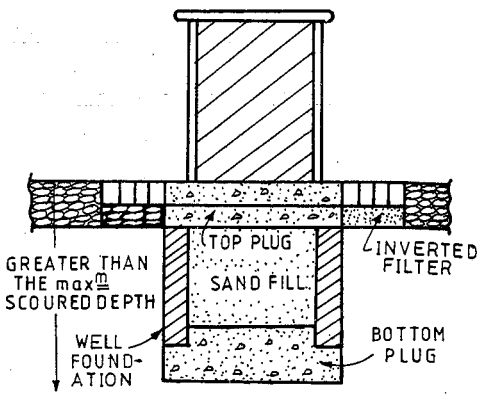


Fig. 9.11. (*b*) Cross-section of Divide Wall beyond Pucca floor.

The main functions served by the divide wall are :

(*a*) It separates the 'under-sluices' from the 'weir proper'. Since the crest level of the under-sluices is lower than that of the weir proper, the two must be separated, and this is being done by the Divide wall.

(*b*) It helps in providing a comparatively less turbulent pocket near the canal head regulator, resulting in deposition of silt in this pocket and, thus, to help in the entry of silt-free water into the canal.

(*c*) Divide wall may keep the cross-currents, if at all they are formed, away from the weir. Cross-currents lead to vortices and deep scours, and therefore, prove hazardous to weirs. For this purpose, additional divide walls projecting at right angle to the weir may sometimes be constructed at equal or desired spacings. They may, then, act as groynes, protecting, in turn, the weir from cross-currents, as shown in Fig. 9.12.

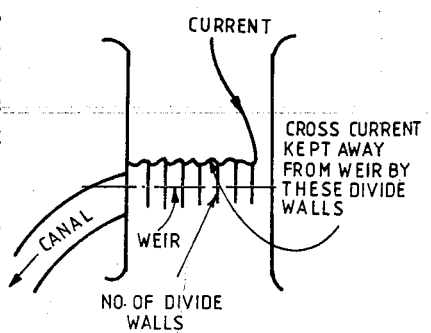


Fig. 9.12

These cross-currents may sometimes develop, when there is a natural tendency for the main river current to attack the bank opposite to

the canal head regulator. In such a case, the water will be forced by the weir to flow towards the head regulator, and cross-current will develop. This function of the divide walls, is not fully established, and the most modern practice is only to construct the divide wall for separating the 'under-sluices' from 'weir proper'.

Design considerations. The divide walls can be designed as cantilever retaining walls subjected to silt pressure and water pressure from the under-sluice side. For the worst case, the design should be checked for full silt pressure on the pocket side with equal water on two sides during low floods, and also for maximum differential water-head, when full discharge is passing through the under-sluices and no discharge is passing the weir. The value of differential pressure, may be taken, arbitrarily, as 1.0 m for waterhead and about 2.0 m for silt pressure.

9.3.5. River Training Works. River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from outflanking the works due to a change in its course. The river training works required on a canal headworks, are :

(*i*) *Guide banks* ;

(*ii*) *Marginal bunds* ; and

(*iii*) *Spurs or groynes.*

The guide banks force the river into a restricted channel, and thus, ensuring a smooth and an almost axial flow near the weir site. The design and other details of the guide banks have already been discussed in the previous chapter.

Marginal bunds are provided on the upstream side of the works in order to protect the area from submergence due to rise in HFL, caused by the afflux. These bunds are, therefore, continued till they join contours higher than the new HFL. Construction of these bunds would be justified only, when the value of land saved is more than the cost of the marginal bunds. In certain cases, when the ponding is high and the watershed is low, the construction of marginal bunds may almost become obligatory. The layout of the marginal bunds is very important in economising the overall cost of the training works and subsequent cost of maintenance. Apart from their primary purpose of protecting the adjoining area from river spills caused by afflux, these bunds do help in partially controlling and guiding the river flow between the guide banks. For this reason, the length of the guide banks do depend on the layout of the marginal bunds.

Marginal bunds are nothing but earthen embankments, protected by groynes, wherever needed. They are designed on the principles of design of earthen dams or dikes. The design aspects of groynes have already been discussed in the previous chapter.

9.3.6. Fish Ladder. Large rivers are generally inhabited by several types of fish, many of which are *migratory*. Such migratory type of fish, called *anadromous fish*, move from one part of the river to another part, according to the season. In India, only one such migratory fish is found, and this specie is known as **Hilsa**. *Salman, Steel head trout,* etc. are the other species of such anadromous fish, found in other countries.

Generally speaking for India, these anadromous fish have been found to be moving from upstream (hills) to downstream (plains) in the beginning of the winter season in search of warmer waters, and return to their spawning grounds upstream, slightly before monsoons, in the month of May and June. If no arrangement is made in the weir or the dam, to enable the fish to pass upstream, then such migratory fish have been found to

be striking and striking against the water current in their efforts to move up, till death. Non-provision of an arrangement for fish to pass upstream, may thus, lead to large scale destruction of fish life.

It has been established that most kinds of fish can travel upstream, against a flow velocity of about 3 to 3.5 m/sec. Usually, there is a head difference of 5 to 6 metres between the upstream of the weir and the downstream water level in the river. If simply an open gap is left in the weir for this migration, the velocity of flow through such an opening will be very high. Therefore, even the strongest fish will not be able to travel upstream ; resulting in large scale destruction of fish near the downstream end of the fish gap. This had actually happened on the river Ganges at old Bhimgoda weir in Hardwar (U.P.), where a suitable arrangement was later on made for this migration.

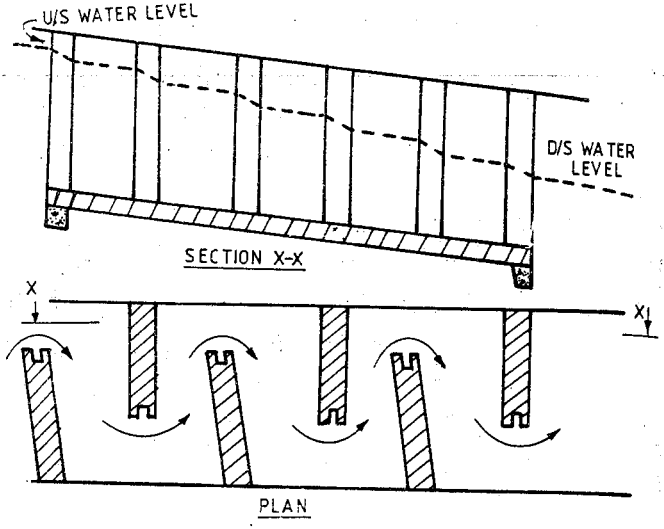


Fig. 9.13. (*a*) Section and plan of a typical fish ladder.



Fig. 9.13. (*b*) Photographic view of a fish ladder.

A structure which enables the fish to pass upstream is called a '*fish ladder*'. It is a device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity, not exceeding 3 to 3.5 m/sec. This object is generally accomplished by providing a narrow opening adjacent to the divide wall and provide suitable baffles or staggering devices in it, so as to control the flow velocity.

Pool type and Steep Channel type are commonly used types of fish ladders in weirs and barrages. Typical sketches (section, plan and photographic view) of ordinarily used **steep channel type** of a fish ladder, are shown in Fig. 9.13 (*a*) and (*b*).

In this type of ladder, the space between the weir and the divide wall, is provided with oblique walls, and holes are also staggered, so that the fish can take rest after passing one hole before they move on to the other. Sometimes, a fish ladder is provided within the divide wall itself, as shown in Fig. 9.14.

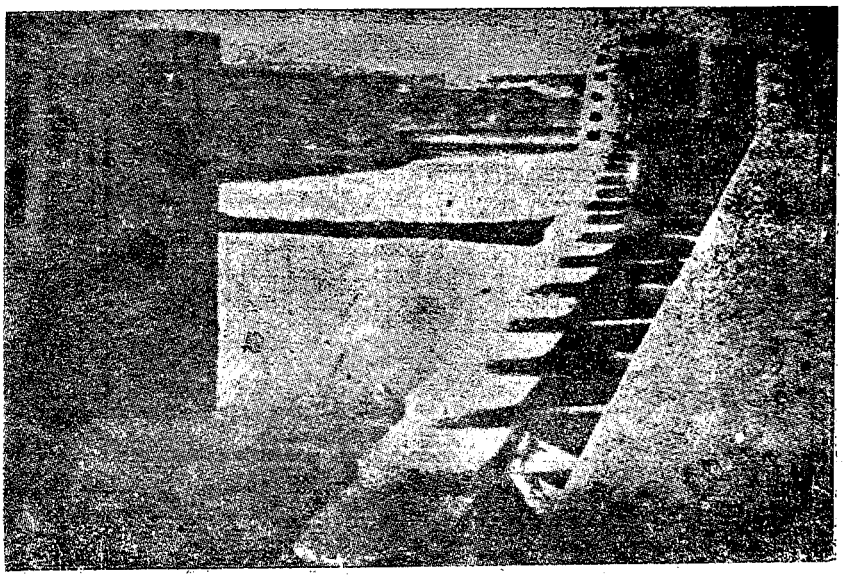


Fig. 9.14. Photographic view of a fish ladder provided in the divide wall.

Fish ladders are generally not found to be economical on dams higher than 100 m or so. For such cases, fish are usually transported upstream by mechanical means, using tramways or tank trucks. In dams, the fish also need to be protected from the turbines and spillways. If the head is less than 5 m or so, the fish can pass through the turbines and spillways with very low mortality.

Various types of screens and louvers are used to divert the fish into a safe by-pass, which may be in the form of a short length of a pressure pipe, through which the fish pass downstream, or the fish may be diverted into a pond, from which they can be extracted for downstream transport by tank trucks.

9.3.7. The Canal Head Regulator or Head Sluices. A canal head regulator (C.H.R.) is provided at the head of the off-taking canal, and serves the following functions :

(*i*) *It regulates the supply of water entering the canal.*

(*ii*) *It controls the entry of silt in the canal.*

(iii) It prevents the river floods from entering the canal.

A typical cross-section of a head regulator is shown in Fig. 9.15. The regulator is generally aligned at right angle to the weir, but slightly larger angles (between 90° to 110°) are now considered preferable for providing smooth entry of water into the regulator, as shown in Fig. 9.16.

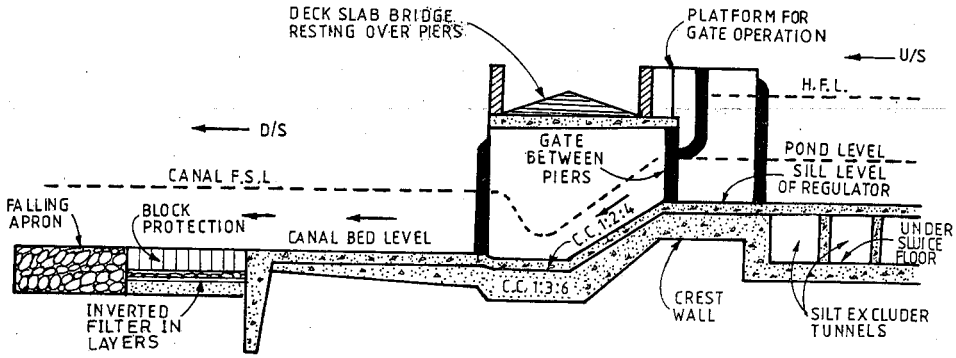


Fig. 9.15. A typical section through a Canal Head Regulator.

The regulator is provided with gates. Smaller spans were used in the past, but with the improvements in gate constructions, larger spans, say between 6 to 10 metres, are generally used these days, if found economical.

The water from the under-sluice pocket is made to enter the regulator bays, so as to pass the full supply discharge into the canal. The maximum height of these gated openings, called **head sluices** will be equal to the difference of Pond Level and Crest Level of the regulator. The entry of silt into the canal is controlled by keeping the crest of the head regulator by about 1.2 to 1.5 metres higher than the crest of the under-sluices. If a Silt-Excluder* is provided, the regulator crest is further raised by about 0.6 to 0.7 metre. Silt gets deposited in the pocket, and only the clear water enters the regulator bays. The deposited silt can be easily scoured out periodically, and removed through the under-sluice openings.

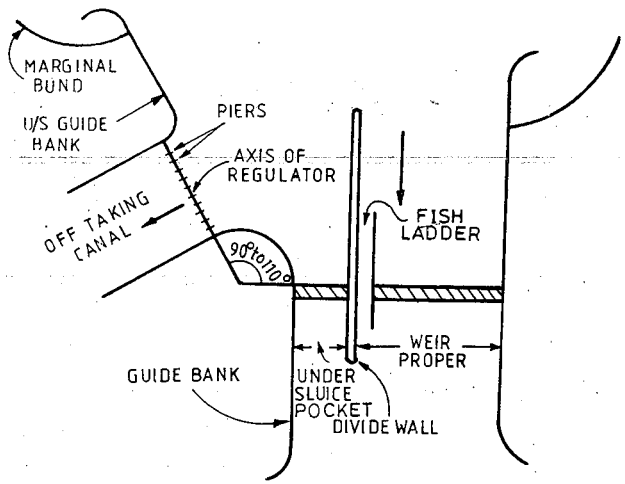


Fig. 9.16. Alignment of a Canal Head Regulator.

The crest level of the regulator, generally called the **sill level,** is not only governed by silt considerations, but is also governed by the discharge considerations. The full supply discharge has to pass through the regulator openings, the height of which will

* Explained in the next article.

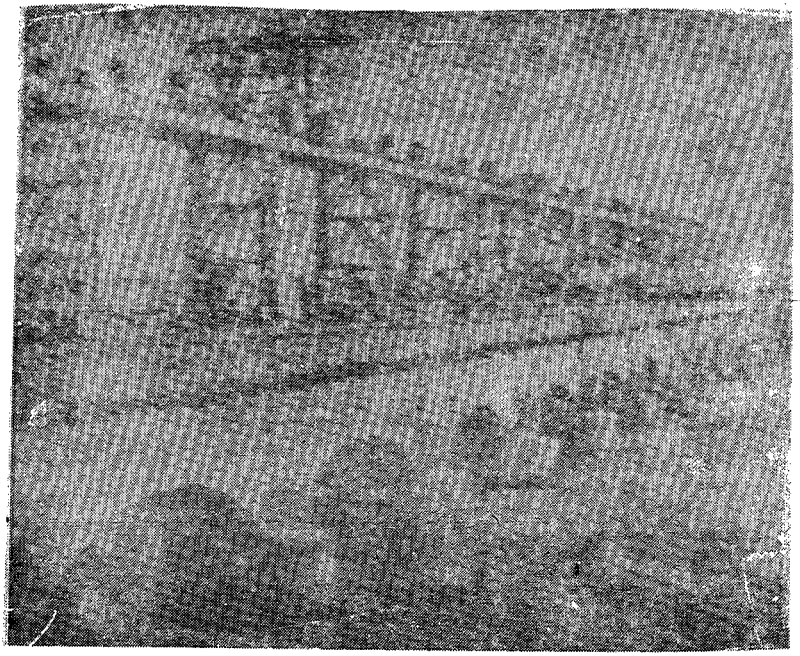


Fig. 9.17. Photographic view of a Canal Head Regulator.

be equal to the difference of pond level and sill level. *The smaller the height of the openings, the larger will be the width of the openings. In other words, if the sill level is raised too much, the clear waterway of the regulator will increase enormously, and, thereby making it more costly.*

The waterway of the regulator is fixed so that the full supply discharge of the canal could easily pass over the crest of the regulator with designed pond level, with ample factor of safety to allow for the future silting up of the canal and the subsequent rise in its FSL. Since a regulator is provided with a very wide and a shallow waterway, the drowned weir formula is used to calculate the discharge, and is given below :

With reference to Fig. 9.18, we have

$$Q_1 = \frac{2}{3} \cdot C_{d_1} \sqrt{2g} \cdot B. \left[(h + h_v)^{3/2} - h_v^{3/2} \right] \text{(free weir equation)} \quad ...(9.1)$$

$$Q_2 = C_{d_2} \cdot B \cdot h_1 \sqrt{2g \cdot (h + h_v)} \text{ (drowned weir equation)} \quad ...(9.2)$$

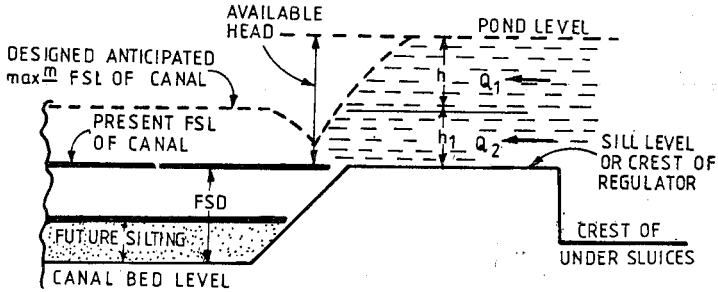


Fig. 9.18

Total discharge $Q = Q_1 + Q_2$

or $$Q = \frac{2}{3} C_{d_1} \cdot \sqrt{2g} \cdot B. \left[(h + h_v)^{3/2} - h_v^{3/2} \right] + C_{d_2} B \cdot h_1 . \sqrt{2g \cdot (h + h_v)} \quad ...(9.3)$$

The values of C_{d_1} and C_{d_2} may be taken as 0.577 and 0.80, respectively.

where h = Difference of the upstream and downstream water level, *i.e.* (Pond level — Maximum anticipated FSL of canal after making due allowance for future silting up of canal).

h is called the utilized head or working head or designed head for the regulator, and is generally taken as half of the available head, where available head is equal to the difference of Pond Level and actual FSL of canal.

h_v = Head due to velocity of approach

B = Clear width of waterway

h_1 = Depth of downstream water level above the crest

i.e. (maximum anticipated FSL of canal minus sill level)

When all other variables are fixed and known, value of clear waterway width (B) can be calculated.

The width of the waterway (B) calculated above, generally works out to be more than the normal width of the canal down-stream. In such a case, the sill level may be lowered, so as to increase the head and to decrease waterway to make it equal to the width of the canal. But the sill level is also governed by silt exclusion considerations, and therefore, many a times, it may not be possible to lower the sill level. In such a case, the calculated value of waterway is provided and the normal required width of the canal is obtained by contracting the wings.

As stated earlier, in a canal head regulator, the gate controlled openings are provided from the sill level to the pond level. During high floods, the water level in the river pocket will, however, be much higher than the pond level. To avoid spilling of this water over the canal regulator gates, a R.C.C. wall is provided from the pond level up to river HFL. This wall spans for the entire length of the regulator and will rest over the piers of the regulator bays. This wall is known as **breast wall,** and will be subjected to vertical self-weight and horizontal water pressure acting against it from the upstream side. Its design may be done as cantilever retaining wall for the horizontal water pressure, and as a girder spanning between the supports for its self-weight.

The length and thickness of horizontal floor, glacis, protection aprons, etc. is worked out on the same principles as are applicable to weir design, explained later in chapter 11. Usually, the worst condition for uplift pressure occurs, when high flood is passing down the weir and there is no flow in the canal.

Example 9.1. *The head regulator of a canal has 3 openings each 3 m wide. The water is flowing between the upper and lower gates. The vertical opening of the gate is*

1.0 m. The head on the regulator is 0.45 m (Afflux). If the upstream water level rises by 0.20 m, find how much the upper gates must be lowered to maintain the canal discharge unaltered. *(Madras University 1976)*

Solution.

The width of regulator openings = 3 spans of 3 m each = 9 m.

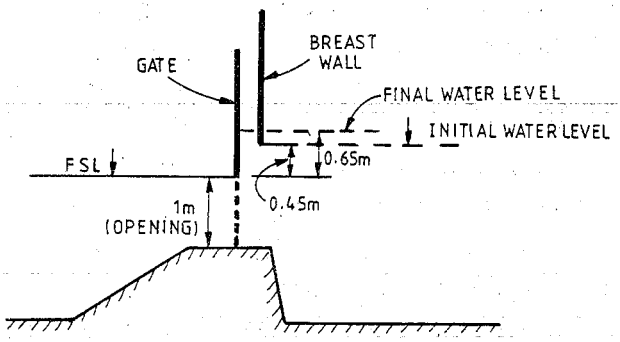


Fig. 9.19

When the gate opening is 1 m. The discharge can be calculated by submerged orifice formula, *i.e.*

$$Q = C_d \cdot A \sqrt{2gh} = 0.62\,(1 \times 9)\sqrt{2 \times 9.81 \times 0.45} \quad ...(i)$$

In the 2nd case, when upstream water level rises by 0.2 m, let the gate opening be x metre to keep the discharge unaltered.

$$\therefore \quad Q = 0.62\,(x \times 9)\sqrt{2 \times 9.81 \times 0.65} \quad ...(ii)$$

Equating (i) and (ii), we have

$$0.62\,(1 \times 9)\sqrt{2 \times 9.81 \times 0.45} = 0.62\,(x \times 9)\sqrt{2 \times 9.81 \times 0.65}$$

or $\quad 1\sqrt{0.45} = x\sqrt{0.65}$

or $\quad 0.45 = x^2\,(0.65)$

or $\quad x = 0.83$ m

Hence, the gates must be lowered by an amount

$$1 - 0.83 = 0.17 \text{ m } \textbf{Ans.}$$

BARRAGE REGULATION AND SILT CONTROL AT HEAD WORKS

The supplies entering a canal which takes off from the upstream side of a weir or a barrage, can be regulated into the following two ways :

(*i*) **Still Pond Regulation Method.** In this method of operation, all the gates of the scouring sluices are kept closed when the canal is running. Only as much discharge is drawn into the under-sluice pocket as much is required for the canal ; the surplus being escaped over some other section of the weir or the barrage. The velocity of water in the under-sluice pocket, therefore, gets reduced, as the smaller discharge enters through the same waterway. The low velocity causes the sediment to settle down and relatively clearer water enters the canal.

The silt is, thus allowed to accumulate in the pocket till it reaches to within 0.5 m below the crest of the regulator. The canal is then closed and the scouring sluices opened, till the entire silt deposit gets washed away on the downstream of river. The scouring

operation takes about 24 hours, and for this much of time, the canal supply has to be stopped. After the silt deposit is washed out, the scouring sluices are closed and the canal supply is restored again. This method of barrage regulation is called still pond regulation and is very useful in controlling the amount of silt entering the canal. However, this method of regulation is practicable only where the sill of the canal head regulator is sufficiently higher than the upstream floor of the under-sluices. Moreover, it leads to closure of the off-taking canal atleast once a month, resulting in the wastage of discharge and loss of irrigation to that much extent.

(*ii*) **Semi-Open flow Operation.** In this method, water in excess of the canal requirement, is allowed to enter into the under-sluice pocket. The gates of the scouring sluices are kept partially opened. The total water entering the pocket, thus, gets divided into two parts, in front of the head regulator. The top water (above the sill level of head regulator) enters the canal through the head regulator, and the bottom water escapes downstream through the under-sluices. Due to this, certain velocity is maintained in the pocket and silt remains in suspension. The turbulence created may sometimes cause even the coarser silt to rise up and to enter into the canal. The silt control, is thus reduced in this method. However, the advantage of this method is that the silt is constantly and continuously scoured out, and the canal has not to be closed, as is required to be done in still pond regulation method.

Even in spite of using still pond regulation techniques, certain amount of silt may enter the canal. And also, silt will definitely enter the canal in semi-open flow operation. In order to avoid this silt, special silt control works are constructed, which remove the silt from the canal water either before the water enters the canal or after the water has entered and travelled a certain distance in the canal. These devices are explained below in details :

9.3.8. Silt Control Devices. The entry of silt into a canal, which takes off from a Head-Works, can be reduced by constructing certain special works, called silt control works. These works may be classified into the following two types :

(*a*) **Silt Excluders.** Silt excluders are those works which are *constructed on the bed of the river*, upstream of the head regulator. The clearer water enters the head regulator and the silted water enters the silt excluder. In this type of works, the silt is, therefore, removed from the water before it enters the canal.

(*b*) **Silt Ejectors.** Silt ejectors, also called **silt extractors,** are those devices which extract the silt from the canal-water after the silted water has travelled a certain distance in the off-take canal. These works are, therefore, constructed on the bed of the canal, and a little distance downstream from the head regulator.

Principles of Silt Control

Before we describe the mechanism and functioning of these silt control devices, we shall explain the basic principle on which the silt is removed from the water.

The fundamental principle behind silt control is : that most of the silt tries to settle down in water, thus, confining itself mostly in the bottom layers of water. We also know that the silt is kept in suspension by the force of the vertical eddies generated by the friction of the flowing water against the bed. In other words, if this bed friction is more, the upward force of eddies shall be more, and hence, lesser chances of silt settlement will exist. Hence, if the friction can be reduced by constructing a smooth approach channel, more settlement of silt and its consequent removal, is possible. Further, the silt from the bottom layers can be removed, by separating the top layers and the bottom

layers, without causing any disturbance to the flow. The chances of less disturbance and that of providing a smooth approach channel can be better attained in a canal rather than in the river bed. Hence, the works *which are constructed in the canal* (*i.e.* the silt extractors or silt ejectors) will *definitely be superior and more effective than the works* which are constructed in the river bed (*i.e.* the silt excluders). *Silt extractor is therefore, better than a silt excluder.* However, the silt extractor shall be costlier because surplus water has to be taken into the canal from the head, and an escape channel which will feed the highly silted water back into the river, shall have to be constructed.

9.3.8.1. Description and Design of a Silt Excluder. A silt excluder consists of a number of rectangular tunnels running parallel to the axis of the head regulator and terminating near the under-sluiced weir. The tunnel nearest to the crest of the head regulator has to be at least of the same length as the head regulator. Other tunnels may be shorter in length. The roof slab of the excluder tunnels is kept at the same level as that of the regulator crest, as shown in Figs. 9.15 and 9.20. The bottom layer of water which is highly charged with silt and sediment will pass down the tunnels and escape

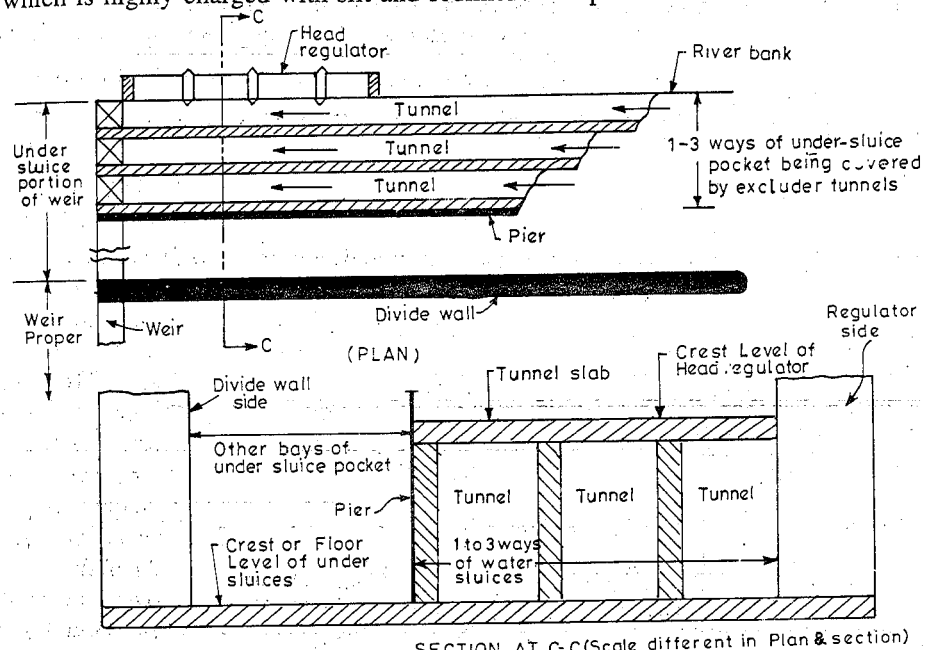


Fig. 9.20. Silt Excluder.

over the floor of the under-sluice way(s), since the gates of the under sluice way(s) shall be kept open upto the top of the tunnels. The clearer water over the top of the roof of the excluder tunnels, will thus enter the canal through the head regulator. Usually, two or three bays of the under-sluices of the weir or the barrage are covered by the excluder. *However, excluder covering only one bay has been designed and is usually adopted for sandy rivers.*

The **major disadvatages of silt excluders** are :

(*i*) Simultaneous use of under sluices-masked by the silt excluder tunnels, for silt exclusion and their assigned purpose of passing high river discharge, cannot be obtained.

(*ii*) The tunnels constructed on the river bed will have to be of robust construction, as to withstand river action.

(*iii*) In the presence of tunnels, on the river bed in the under-sluice pocket, securing of good approach conditions become difficult. If these tunnels were not there, a curved off-take or a curved approach could be provided at a small cost.

The **major advantages of the pocket excluders** are :

(*i*) As the tunnels are large, there are unlikely to be choked by rolling or suspended debris.

(*ii*) Since the same gates of the scouring sluices can be utilised for the excluder tunnels, some economy in cost is secured.

(*iii*) At the weir, sufficient head for operating the excluder is generally available.

Design of tunnels. A theoretical design of a silt excluder is confined mainly to find out the area of the tunnel openings required to pass the designed discharge and to determine its structural requirements. On the basis of past experience and model studies, the design of an excluder is finalised. The Punjab Irrigation Research Institute has carried out extensive research in the design of excluders. Based on their research findings, it has been found that if the excluder discharge is restricted to 15 to 20% of the canal discharge, satisfactory silt exclusion can be obtained. A minimum velocity of 2 to 4.5 m/sec must be maintained through the tunnels in order to keep them free from sediment. A value of 3 m/sec is usually adopted for ordinary straight reaches. A higher value of 4 to 4.5 m/sec may be adopted for boulder-stage rivers. A lower value of about 2 m/sec may be adopted for sandy rivers. After fixing the discharge and velocity, the cross-sectional area of the excluder tunnel-openings, can be determined. Knowing the height, the required width can be found and divided into a suitable number of bays.

By model studies, it has been found that the tunnels should be located at selected positions, rather than distributed uniformly over the entire length of the regulator. The position of tunnels being more important than their number. River curvature must be taken into consideration as far as possible. It was found by experience at Khanki that three tunnels were more efficient than six. Also, a smaller number of pocket bays covered by the excluder, give better results as do the openings of the tunnels confined to the mouth. Side openings into the tunnels, have been found to decrease the efficiency. Usually, 4 to 6 tunnels are generally provided.

Approach and exit. At the entrance, the tunnels are generally given a bell mouthed shape so as to increase the zone of suction. The radius of bell mouthing varies from 2 to 6 times the tunnel width ; the radii increasing for tunnels away from canal head regulator and for shorter tunnels. At the exit end, the tunnels are throttled for restricting the discharge to the desired value and to increase the velocity to prevent deposition of silt.

Height of tunnels. The height of tunnels generally varies from 0.5 to 0.6 m for sandy rivers, and 0.8 to 1.2 m for boulder stage rivers.

Body and roof of tunnels. The roof of tunnels must be strong enough to withstand the full water load coming from its top, with no water inside. Such a condition may arise when the tunnels are closed from upstream for repairs or for some other purpose. Moreover, the tunnel walls and roofs must be strong enough, as not to be, damaged by debris, boulders, shingles, etc., during floods. This is all the more important, since

damage to excluder is not discovered as they are always under water and difficult to repair. Moreover, even the slight damage to the tunnel roof has found to have made the excluder less efficient.

Outfall channel. It should also be ensured that a channel to take the silt laden water into the river is maintained downstream of the excluder. No separate outfall channel is generally required for sandy rivers as a continuous flow through the alluvial bed, itself makes a channel there. However, if gravel and boulders are to be excluded and the main channel is away, a lined channel may be necessary, or a channel may have to be created by suitable training measures. This was done in the case of Tajewala Head Works on Yamuna river, where two short spurs induced the river to flow on the side of the under-sluices, and thus eliminated the construction of a separate outfall channel.

Losses in tunnels. The following head losses may be taken into account for calculating the tunnel sections :

(*i*) *Frictional Loss.*

$$h_f = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \text{ (Manning's formula).} \qquad ...(9.4)$$

where h_f = Head loss in metres.
L = length of tunnel in metres.
R = Hydraulic mean depth in metres.
n = Manning's Rugosity coefficient.

(*ii*) *Loss due to Bend.*

$$h_b = F\left(\frac{V^2}{2g}\right)\left(\frac{\theta}{180°}\right) \qquad ...(9.5)$$

where $F = 0.124 + 3.134\left(\frac{S}{2r}\right)^{1/2}$ ...(9.6)

θ = Angle of deviation in degrees
S = Width of tunnel in metres.
r = Radius of bend along centre line of tunnel in metres.
h_b = Head loss in metres.

(*iii*) *Transitional loss due to change of velocity in contraction.*

$$h_c = 0.1\left[\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right] \qquad ...(9.7)$$

where V_1 = Velocity in canal upstream in m/sec.
V_2 = Velocity in tunnel in m/sec
h_c = Head loss in metres.

(*iv*) *Transitional loss due to change of velocity in expansion.*

$$h_e = 0.1\left[\frac{V_2^2}{2g} - \frac{V_3^2}{2g}\right] \qquad ...(9.8)$$

where V_2 = Velocity in tunnel.
V_3 = Velocity downstream of tunnel.

9.3.8.2. Description and Design of a Silt Extractor or a Silt Ejector. The typical layout of a silt ejector is shown in Fig. 9.21 (*a* and *b*). It essentially consists of a horizontal diaphragm slab, a little distance above the canal bed, which separates out the bottom layers. Under the diaphragm, which is normally spanning the entire width of the

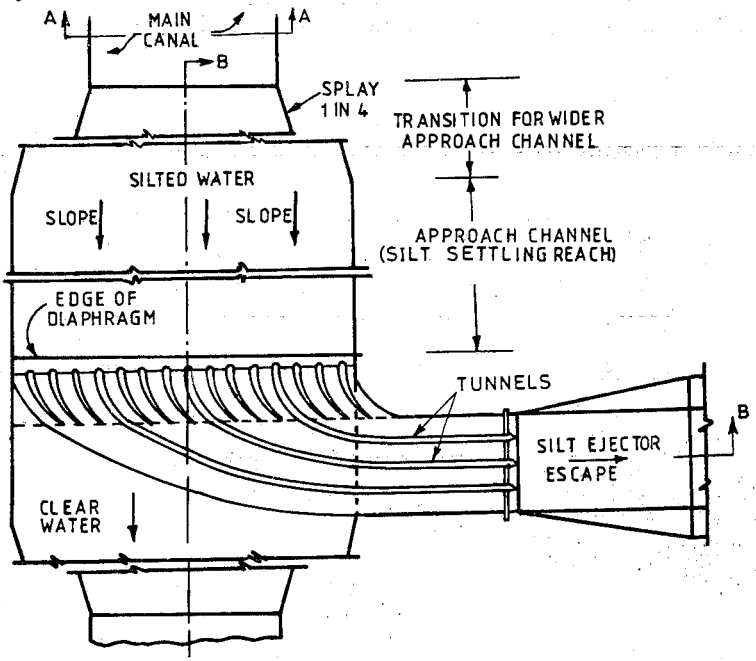


Fig. 9.21. (*a*) Plan of Silt Ejector.

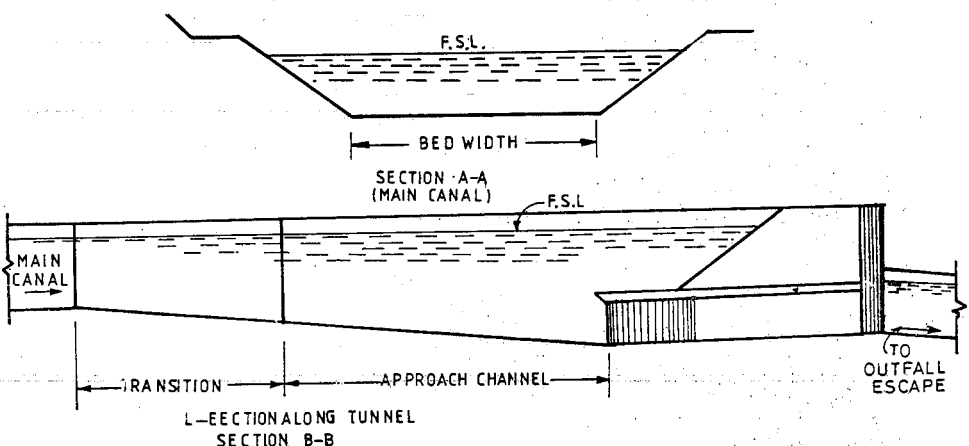


Fig. 9.21. (*b*) Sections of a Silt Ejector.

canal, there are tunnels or compartments which extract the highly silted bottom water into an escape channel. The tunnel entrances should be designed in such a way that there is no disturbance of flow at entry, and the escape flow is quickly accelerated under the diaphragm, so as to prevent the clogging of tunnels. These objectives are achieved by dividing the entire span width into a number of compartments or tunnels by means of

streamlined vanes, which are gradually converging (thus gradually decreasing the sectional area and correspondingly increasing the flow velocity). These main compartments are sub-divided into smaller compartments or sub-tunnels by vanes of radii varying from 3 to 4 times the width of the sub-tunnels to avoid cross flow in the transition section. The tunnel discharge is regulated by means of gates at the outlet end. The escape channel should have sufficient slope to carry the silt laden water without getting silted up, and should lead the water by the most economical path back to the river. The principles governing the location and design of various components of a silt ejector are briefly described here.

Location. The silt ejector should be located in the canal neither too much near the head regulator nor too far away from the head regulator. In the former case, the residual turbulence may keep the silt in suspension, and thus, preventing its extraction up to the desired extent. Similarly, in the latter case, when the ejector is located far away from the head regulator, the silt may settle down earlier and reduce the channel capacity upstream, thus, defeating the very purpose of the ejector.

Design Principles for Various Components are given below :

(*i*) **Approach channel.** The normal main canal section is generally widened after the canal takes off from the head regulator, so as to reduce the flow velocity to a desired level. This will help in increasing the silt concentration in the bottom layers. The reduced velocity should be maintained for a sufficient length to achieve the desired sediment concentration in the bottom layers.

(*ii*) **Diaphragm.** The diaphragm should be placed in such a way that it causes least disturbance in front of the ejector tunnels, so as not to disturb silt concentration attained in the bottom layers. The diaphragm is generally placed at the downstream bed level of the canal, *i.e.* the canal bed has to be slightly depressed under the diaphragm. However, if the diaphragm has to be placed higher due to some other considerations, the condition of fall, particularly for low supplies, should be checked, and energy dissipation arrangements made, if found necessary. The diaphragm should be properly tied to the supports, so as to prevent it from being dislodged. The diaphragm shall be extended beyond the pier noses and the underneath of the diaphragm shall be given a streamlined bell mouth shape confirming to the equation of an ellipse, *i.e.*

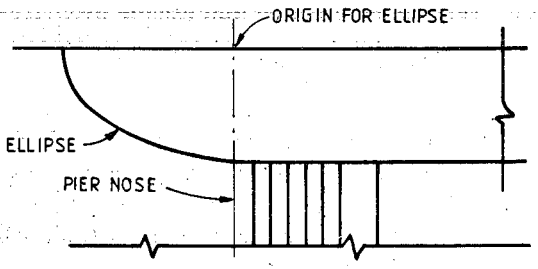


Fig. 9.22

$$\frac{x^2}{4a^2} + \frac{y^2}{a^2} = 1$$

where a is the thickness of the diaphragm. (See Fig. 9.22).

(*iii*) **Tunnels.** The tunnels or compartments shall be constructed by gradually converging vanes, as explained earlier. The upstream noses of the vanes and piers may have cut-water shapes, while the downstream end of vanes may be fish-tailed. The tunnel dimensions at the entry and exit may be so fixed, as to ensure velocities that would carry the size of sediment to be removed. The section of the sub-tunnels at the entry may be

so chosen that the velocity of flow at the intake is slightly higher than the velocity of bottom filaments of water upstream of ejector. The section of sub-tunnels up to their exit, where they end into the main tunnels, may be reduced gradually in such a way that there is an overall increase of 10 to 15 per cent in the velocity of emerging flow.

At the exit of sub-tunnels, the section of the main tunnel may be designed such that the flow velocities of the combined discharge are not less than the velocities emerging from the sub-tunnel. The section at the exit of main tunnels may be so designed, as to attain a velocity 2.5 to 6 m/sec depending on the grade of sediment to be ejected ; but in all cases, the exit velocity has to be more than the critical velocity, so as to ensure super-critical flow. The height of the tunnels is generally kept from 0.5 to 0.75 metres.

The width of the tunnels may be so adjusted as to have equal losses in each tunnel.

Note. As the lengths of tunnels are different, the head loss in them would be different, if all of them were to take the same discharge. As the difference between the water level in the canal and that in the escape channel is the same, the head available for all the tunnels is the same. Hence, the shorter tunnel would have a higher velocity of flow through it, than in the others, and this would lead to disturbance at entry. To counteract this possibility, either the shorter tunnel may serve a larger width of the canal than a longer one, or the tunnel cross-sections may be altered, each serving an equal width. The adjustment of width is recommended by BIS.

(*iv*) **Gate control regulation.** The discharge from ejector is controlled by gated regulation at the downstream end of the tunnels. The amount of discharge passing down the ejector and the frequency of its operation would vary in different parts of the year depending on the silt load carried by the canal. Proper gate regulation is, therefore, required, as per the intelligence and initiative of the maintenance engineers. The gate must be operated occasionally, so as to ensure it to be in working order.

(*v*) **Escape channel and escape discharge.** The outflow from the ejector is taken to a natural drainage through an escape channel. The escape channel should be designed to have a self-cleansing velocity, so that the ejected material is transported without deposition. Adequate fall between the F.S.L. of the escape channel at its tail end and the normal H.F.L. of the natural drain, is desirable for efficient functioning of the channel.

The escape discharge is generally governed by : (*i*) amount of discharge required to remove the desired sediment size and load ; and (*ii*) minimum discharge required for flushing individual tunnels. Normally, an escape discharge varying from 10 to 20% of the full supply discharge of the canal, downstream of the ejector, is sufficient for this purpose.

Efficiency of Excluders and Ejectors

The efficiency of excluders and ejectors, can be expressed by

$$\eta = \frac{I_u - I_d}{I_u} \times 100 \text{ (per cent)} \qquad ...(9.9)$$

where η = Per cent efficiency.

I_u = Silt intensity in canal upstream of the ejector.

I_d = Silt intensity in canal downstream of the ejector.

PROBLEMS

1. Draw a neat sketch of a River-regulator and explain its salient components.

Write short notes on :

(*i*) Silt ejector. (*ii*) Scouring sluices.

(*iii*) Dividing groyne. (*Madras University, 1974*)

(**Hint** : Dividing groyne means divide wall.)

2. (*a*) Differentiate between a weir and a barrage.

(*b*) Draw a neat layout of Diversion head-works and indicate the various components of the system. Briefly indicate the function of each component. (*Madras University, 1973*)

3. (*a*) What is the purpose served by "scouring sluices" at weirs ? What are the important points to be borne in mind in designing these sluices ? (*Madras University, 1975*)

(*b*) How does "scouring-sluices" differ from those of "head-sluices" ?

(*c*) Why is it necessary to provide a "fish ladder" on large rivers, and how does it help in achieving the required objective ?

4. Draw a neat layout of a river regulator, and indicate suitable locations for the following :

(*i*) A navigational lock (*ii*) A fish ladder

(*iii*) Scouring sluices (*iv*) Head sluices

Draw a typical cross-section of a head regulator, and indicate the various components of the same. (*Madras University, 1973*)

5. What are the functions of a canal head regulator ?

Design a head sluice for a canal, intended to irrigate 24,000 hectares at a duty of 800 hectares per cumec. It is in a main canal taken from a river and has the following data :

Head available to permit full supply in canal = 0.30 m.

Bed width of canal = 20 m.

FSD in canal = 1.5 m

Depth in the river at max. water level = 4.2 m (*Madras University, 1975*)

6. Enumerate and explain briefly the different methods for control of entry of silt into canals. Give a neat sketch of any one silt exclusion work to show its structural details and principle of functioning.

Show the component parts of a diversion head-works on a neat sketch. Mention the functions and important design considerations pertinent to the divide wall.

7. (*a*) Illustrate with neat sketch the following parts of a barrage :

(*i*) Sheets piles (*ii*) Fish ladder

(*iii*) Silt excluder (*iv*) Divide wall.

(*b*) Briefly describe a system of regulation that is practised to minimise sediment load entering a canal. (*A.M.I.E., 1975*)

8. (*a*) Differentiate between the following :

(*i*) A barrage and a dam.

(*ii*) Scouring sluices and head sluices.

(*iii*) Surplus weir and storage weir.

(*iv*) Gravity and non-gravity weirs.

(*v*) Silt excluders and silt ejectors.

(*vi*) River regulator and a canal regulator. (*Madras University, 1976*)

(*b*) What is meant by "afflux", and how does it effect the design of weirs and barrages ?

9. (*a*) What is the difference between a weir and a barrage ? Why does a barrage preferred to a weir in modern days ?

(*b*) How does a diversion weir aligned ? Draw a neat sketch showing the different components of a diversion weir scheme.

(*c*) What are the different construction materials which may be used for weirs, and how are the weirs classified on this score ?

10. (*a*) What is meant by an "Intake weir" and how does it differ from a "surplus weir" ?

(*b*) What are "under-sluices", and what are their functions in a river regulator ?

(*c*) Draw a neat sketch of a typical fish ladder indicating the purposes served by it.

11. (*a*) What are "Divide walls", and how do they help in a diversion weir scheme ? Draw neat sketches showing the cross-section of divide walls on pucca floor as well as beyond the pucca floor. Also discuss the design considerations that are involved in designing divide walls.

(*b*) Discuss the use of 'guide banks' and 'marginal bunds' in a river regulator scheme.

12. (*a*) What are the two principal methods of regulating the canal supplies in a diversion head works scheme ? What are their comparative merits and demerits ?

(*b*) Differentiate between a 'Silt extractor' and a 'Silt excluder'. Draw a neat sketch and discuss the principles involved in designing the different components of a silt extractor.

13. (*a*) Differentiate between the following, indicating the preference, which you will give to one, in modern days, with proper reasoning :

(*i*) Weir and barrage

(*ii*) Silt excluder and silt-ejector

(*iii*) Rock-fill weirs and sloping glacis concrete weirs.

(*iv*) Still pond regulation and semi open flow operation at a barrage.

(*b*) Draw a neat sketch of a silt excluder and indicate briefly the principles involved in designing its different components. How will you express its efficiency ?

14. Describe briefly some of the effects of silting in rivers.

Describe briefly a method of removal of silt accumulation behind a river regulator.

(*Madras University, 1973*)

10

Hydraulic Jump and its Usefulness in the Design of Irrigation Structures

10.1. General

Hydraulic jump is the jump of water that takes place when a super-critical flow changes into a sub-critical flow. When a stream of water moving with a high velocity and low depth (*i.e.* supercritical flow) strikes another stream of water moving with a low velocity and high depth (*i.e.* sub-critical flow), a sudden rise in the surface of the former takes place. This phenomenon is called *Hydraulic jump*, and is generally accompanied by a large scale turbulence, dissipating most of the kinetic energy of super-critical flow. Such a phenomenon may occur in a canal below a regulating sluice, at the bottom of a spillway, or at a place where a steep channel slope suddenly turns flat.

When water falls over a spillway or a vertical or a glacis fall, it acquires a lot of momentum and velocity. This high velocity, if allowed to persist, shall cause large scale erosion and scouring of the downstream soil of the work, and hence, it must be checked or controlled. In such situations, the phenomenon of hydraulic jump can be used with great advantage for dissipating the kinetic energy of the water.

If the jump is low, *i.e.* the change in the depth is small, the water shall not rise abruptly but will pass through a number of undulations. Such a low jump is called *an undular jump.* In this case, the energy dissipation shall be low and incomplete. But on the other hand, if the jump is high, *i.e.* when the change in depth is large, the jump shall occur abruptly and is called a *direct jump.* Such a jump involves large scale dissipation of energy and is of considerable importance in the design of hydraulic-cum-irrigation structures.

It may be noted that the depth before the jump is always less than the depth after the jump. The depth before the jump is called the **initial depth** (y_1) and the depth after the jump is called the **sequent depth** (y_2). These depths are shown on specific energy curve (Fig. 10.1) and must be differentiated from *alternate depths* y_1 and y_2'. y_2' is the depth that shall occur in a sub-critical flow if there was no loss of energy in the jump formation ; while y_2 is the actual depth that occurs after the jump, involving the energy loss H_L.

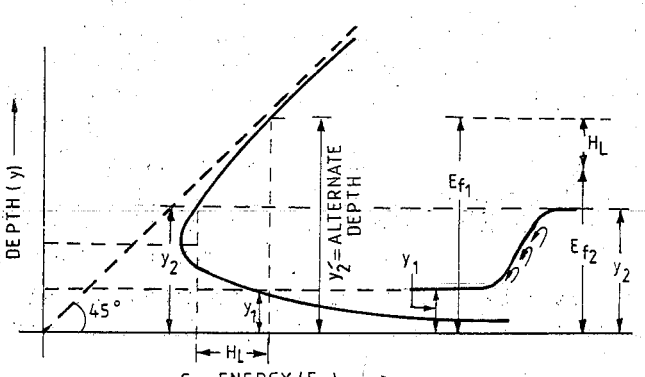


Fig. 10.1. Hydraulic jump interpreted by specific energy curve.

10.2. Types of Jump

Depending upon the incoming Froude No. F_1, the jump on a horizontal floor can be classified, as follows :

(*i*) For $F_1 = 1$, the flow is critical and hence, no jump can form.

(*ii*) For $F_1 = 1$ to 1.7, the water surface shows undulations and the jump is called *undular jump.*

(*iii*) For $F_1 = 1.7$ to 2.5, a series of small rollers develop on the surface of the jump, but the down-stream water surface remains smooth. The velocity throughout is fairly uniform and the energy loss is low. This jump is called a *weak jump.*

(*iv*) For $F_1 = 2.5$ to 4.5, there is an oscillating jet entering the jump bottom to surface and back again with no periodicity. Each oscillation produces a large wave of irregular period, which, very commonly in canals, can travel for miles, doing unlimited damage. This jump is called an *oscillating jump.* The *value of incoming Froude number for barrages and canal regulators generally lies in this zone* or in the previous zone *i.e.* zone of weak jump. The energy dissipators and stilling basins for such conditions need careful design. Different States and Organisations have recommended standard design criteria for the design of stilling basins. A description of these standard basins is available in I.S : 4997—1968. A little description of these stilling basins is given in chapter 20.

(*v*) For $F_1 = 4.5$ to 9.0, the jump is well balanced and its performance is at its best. The energy dissipation ranges from 45 to 70%. The jump is called a *steady jump.*

(*vi*) For $F_1 = 9.0$ and larger, it is a *strong jump* and the energy dissipation may reach up to 85%.

The ranges of Froude number given above, are not definite and may overlap to a certain extent depending on local conditions.

In the subsequent pages, we shall explain the mathematical treatment of hydraulic jump phenomenon on a level surface as well as on a sloping glacis. The methods of plotting the hydraulic jump profiles are also given.

10.3. Momentum Formula

In Fig. 10.2, a stream of water passing through a hydraulic jump has its velocity, depth and energy of flow as V_1, y_1 and E_{f_1} respectively before the jump, and V_2, y_2 and E_{f_2} after the jump. Let H_L be the loss of energy in the jump.

Then $H_L = E_{f_1} - E_{f_2}$.

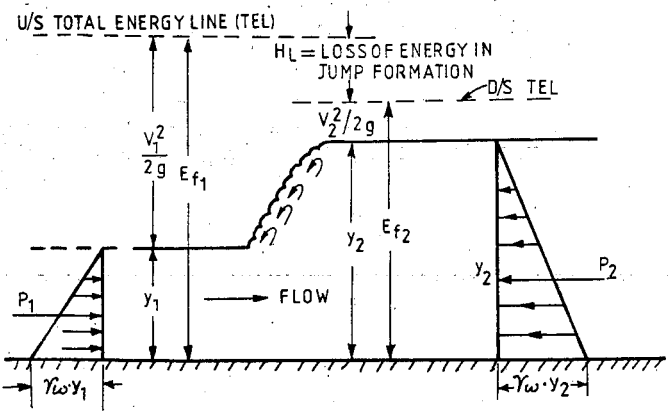
By equation of continuity, we can say that the discharge passing per unit width of the water-way (q) is given as :

$$q = V_1 \cdot y_1 = V_2 \cdot y_2$$

or $$V_1 = \frac{q}{y_1} \qquad ...(10.1)$$

and $$V_2 = \frac{q}{y_2} \qquad ...(10.2)$$

According to Newton's second law of motion, the rate of change of momentum must be equal to the applied force.



U/s
Hydrostatic Pressure
Distribution at section u/s
$P_1 = \frac{1}{2}\gamma_w \cdot y_1^2$

D/s
Hydrostatic Pressure
Distribution at section d/s
$P_2 = \frac{1}{2}\gamma_w \cdot y_2^2$

Fig. 10.2.

Mass of water flowing per second

$$= \frac{\gamma_w q}{g}, \qquad \text{where } \gamma_w \text{ is the unit wt. of water.}$$

Change of momentum per second

$$= \frac{\gamma_w q}{g}\left[V_1 - V_2\right] \qquad ...(10.3)$$

Difference in the total pressure on the two sides of the jump

$$= P_2 - P_1 = \frac{1}{2}\gamma_w y_2^2 - \frac{1}{2}\gamma_w y_1^2$$

$$= \frac{1}{2}\gamma_w (y_2^2 - y_1^2) \qquad ...(10.4)$$

Equating (10.3) and (10.4), we get

$$\frac{\gamma_w q}{g}\left[V_1 - V_2\right] = \frac{\gamma_w}{2}\left[y_2^2 - y_1^2\right] \qquad \text{or} \qquad y_2^2 - y_1^2 = \frac{2q}{g}\left[V_1 - V_2\right]$$

Now substituting $V_1 = \frac{q}{y_1}$ and $V_2 = \frac{q}{y_2}$, we get

$$y_2^2 - y_1^2 = \frac{2q}{g}\left[\frac{q}{y_1} - \frac{q}{y_2}\right] = \frac{2q^2}{g}\left[\frac{y_2 - y_1}{y_1 y_2}\right]$$

or $$y_1 y_2 (y_2 + y_1) = \frac{2q^2}{g} \qquad ...(10.5)$$

or $$y_1 y_2^2 + y_1^2 y_2 = \frac{2q^2}{g} \qquad \text{or} \qquad y_2^2 + y_1 y_2 - \frac{2q^2}{g y_1} = 0$$

or $$y_2 = \frac{-y_1 \pm \sqrt{y_1^2 + \frac{8q^2}{g y_1}}}{2}$$

Neglecting unfeasible – ve sign, we get

$$y_2 = \frac{-y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} \qquad ...(10.6)$$

$$y_2 = \frac{y_1}{2}\left[-1 + \sqrt{1 + \frac{8q^2}{gy_1^3}}\right] = \frac{y_1}{2}\left[-1 + \sqrt{1 + \frac{8V_1^2 y_1^2}{gy_1^3}}\right]$$

$$= \frac{y_1}{2}\left[-1 + \sqrt{1 + 8\frac{V_1^2}{gy_1}}\right]$$

$$y_2 = \frac{y_1}{2}\left[-1 + \sqrt{1 + 8F_1^2}\right] \qquad ...(10.7)$$

where F_1 = Incoming Froude number $= \frac{V_1}{\sqrt{gy_1}}$

Similarly, we can show that

$$y_1 = \frac{y_2}{2}\left[-1 + \sqrt{1 + 8F_2^2}\right] \qquad ...(10.8)$$

where F_2 = Outgoing Froude number $= \frac{V_2}{\sqrt{gy_2}}$

The assumptions made in the derivation of the above formula are :

(*i*) The jump takes place abruptly.

(*ii*) Flow remains streamlined throughout.

(*iii*) Friction is negligible.

(*iv*) Bed is horizontal.

However, in the case of a sloping glacis, the bed does no longer remain horizontal, and the horizontal component of the weight of flowing water will also be involved besides P_1 and P_2 in evaluating the applied force, as shown in Fig. 10.3.

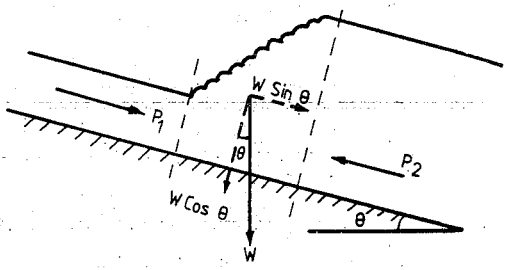


Fig. 10.3

In this case, applied force shall be given by $[P_2 - (P_1 + W \sin \theta)]$ in place of $P_2 - P_1$ in equation (10.2) and hence, the equation derived above for y_2 and y_1 shall not hold good precisely. But if the slope of the glacis is not too pronounced, the gravity component $W \sin \theta$ can be neglected on the grounds that the error so introduced is practically cancelled by the errors involved in other assumptions. Hence, the equations derived for a horizontal bed may be applied to a sloping glacis also, within the limits of accuracy.

10.3.1. Loss of Energy (H_L) in the Standing Wave

$$H_L = E_{f_1} - E_{f_2}$$

where $E_{f_1} = y_1 + \frac{V_1^2}{2g}$ ...(10.9)

$$E_{f_2} = y_2 + \frac{V_2^2}{2g} \qquad ...(10.10)$$

$$\therefore \quad H_L = \left(y_1 + \frac{V_1^2}{2g}\right) - \left(y_2 + \frac{V_2^2}{2g}\right)$$

$$= (y_1 - y_2) + \frac{1}{2g}(V_1^2 - V_2^2) = (y_1 - y_2) + \frac{1}{2g}\left[\frac{q^2}{y_1^2} - \frac{q^2}{y_2^2}\right]$$

or $$H_L = (y_1 - y_2) + \frac{q^2}{2g}\left[\frac{y_2^2 - y_1^2}{y_1^2 y_2^2}\right] \qquad ...(10.11)$$

From equation (10.5), we get

$$y_1 y_2 (y_2 + y_1) = \frac{2q^2}{g} \qquad \text{or} \qquad \frac{q^2}{2g} = \frac{y_1 y_2}{4}(y_2 + y_1)$$

Substituting this value in equation (10.11) we get

$$H_L = (y_1 - y_2) + \frac{y_1 y_2}{4}(y_2 + y_1)\left(\frac{y_2^2 - y_1^2}{y_1^2 y_2^2}\right) = (y_2 - y_1)\left[\frac{(y_2 + y_1)^2}{4y_1 y_2} - 1\right]$$

$$= (y_2 - y_1)\left[\frac{(y_2 + y_1)^2 - 4y_1 y_2}{4y_1 y_2}\right] = (y_2 - y_1)\left[\frac{(y_2 - y_1)^2}{4y_1 y_2}\right]$$

or $$\boxed{H_L = \frac{[y_2 - y_1]^3}{4y_1 y_2}} \qquad ...(10.12)$$

Usually, in any hydraulic jump, the following eight variables are involved : E_{f_1}, V_1, y_1, E_{f_2}, V_2, y_2, q and H_L. These variables are related by six independent equations, as given below :

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} \qquad ...(10.5)$$

$$H_L = \frac{(y_2 - y_1)^3}{4y_1 y_2} \qquad ...(10.12)$$

$$E_{f_1} = y_1 + \frac{V_1^2}{2g} \qquad ...(10.9)$$

$$E_{f_2} = y_2 + \frac{V_2^2}{2g} \qquad ...(10.10)$$

$$V_1 = \frac{q}{y_1} \qquad ...(10.1)$$

$$V_2 = \frac{q}{y_2} \qquad ...(10.2)$$

Hence, if any two variables are known, the remaining six can be worked out by using these six equations, mathematically. The mathematical solution is complicate and to avoid large scale calculations, *Blench* has given some curves by taking q and H_L as known variables (as in actual problem, the discharge intensity q *and the drop in the total energy level* H_L *are generally known*).

Blench had given curves, relating H_L and E_f for different values of q (Plate 10.1). These curves are very useful in determining the location of the jump on a sloping glacis, as explained below.

When Blench curves are not available, the following mathematical equations can be used to evaluate y_1 and y_2 by known values of q and H_L :

Compute y_c = critical depth = $\sqrt[3]{\frac{q^2}{g}}$ ...(10.3)

Now express $\frac{y_1}{y_c} = X$ (say) ...(10.4)

$$\frac{y_2}{y_c} = Y \text{ (say)} \quad ...(10.5)$$

$$\frac{H_L}{y_c} = Z \text{ (say)} \quad ...(10.6)$$

where H_L and y_c are known, hence Z is known.

Relations between Z and X ; and Z and Y are worked out as :

$$Z = \frac{-X^6 + 20X^3 + 8 - (X^4 + 8X)^{3/2}}{16X^2} \quad ...(10.7)$$

$$Z = \frac{-Y^6 - 20Y^3 - 8 - (Y^4 + 8Y)^{3/2}}{16Y^2} \quad ...(10.8)$$

For different values of X and Y, values of Z can be tabulated, and curves $X - Z$ and plotted, to read value of X and Y for known value of Z.

To obtain more direct solution, Swamee C.P, has obtained an approximate solution $-Z$ curves, and related Y with Z by the eqn. :

$$Y = 1 + 0.93556Z^{0.368} \text{ for } Z < 1 \quad ...(10.9)$$

$$Y = 1 + 0.93556Z^{0.240} \text{ for } Z > 1 \quad ...(10.10)$$

From the known value of Z, value of Y *i.e.* y_2/y_c can be calculated to finally compute alue of y_2.

With known values of Z and X, value of X *i.e.* y_1/y_c can also be computed by using qn :

$$Z = \frac{(Y - X)^3}{4XY} \quad ...(10.11)$$

This computed value of X *i.e.* y_1/y_c can be used to compute y_1.

Values of E_{f_1} and E_{f_2} can also be computed by using the values of X and Y, as follows :

$$\frac{E_{f_1}}{y_c} = \varepsilon = X + \frac{1}{2X^2} \quad ...(10.12)$$

$$\frac{E_{f_2}}{y_c} = \eta = Y + \frac{1}{2Y^2} \quad ...(10.13)$$

Location and Profile of the Jump on a Sloping Glacis

10.4.1. Position of the Jump. Let water be passing over the crest of a work (such as a etc.) with a certain head. Then discharge per unit width = $C_d H^{3/2}$, where H is the head ured from the total energy line to the crest and C_d is the coefficient of discharge. Knowing e level of u/s TEL is known. For the given discharge, the depth of water on the d/s is n from gauge discharge curves of the channel. This fixes TEL on the d/s. The difference levels of u/s TEL and d/s TEL gives H_L.

Knowing q and H_L, E_{f_2} can be obtained from Blench Curves (Plate 10.1). Subtracting om d/s TEL, the level at which the jump will form can be easily obtained and hence the

position of point P is fixed. This is how the position of the jump on the sloping glacis is obtained.

Knowing E_{f_2}, E_{f_1} can also be calculated by using, $E_{f_1} - E_{f_2} = H_L$. The corresponding values of y_1 and y_2 for the known values of E_{f_1} and E_{f_2} can be directly read from the 'Energy of Flow Curves' given by Montague (Plate 10.2).

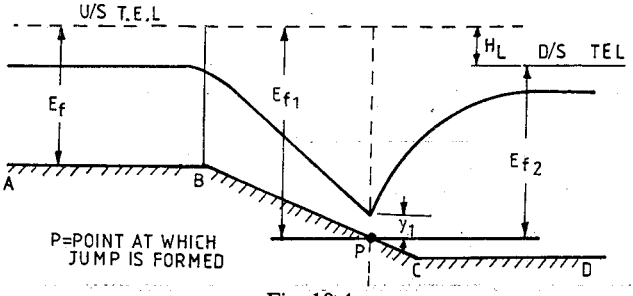


Fig. 10.4

10.4.2. Profile Before the Jump. The water surface profile before the point of jump formation (P) can be easily plotted with the help of Montague's curves (Plate 10.2) as follows : For different points on the glacis from B to P, E_{f_1} will vary, being equal to u/s TEL minus glacis level at the considered point. Since glacis level is different at different points in the length BP, the E_{f_1} will be different at different points and shall go on increasing. For these different values of E_{f_1}, different corresponding values of y_1 can be tabulated from Montague's Curves (Plate 10.2). These values of y_1 shall go on reducing till the point P is reached. These values can be plotted over the glacis, and hence, water surface profile before the jump can be plotted easily.

10.4.3. Profile after the Jump. To plot the water surface profile after the jump point (P), it is necessary to know the Incoming Froude No. F_1.

$$F_1 = \frac{V_1}{\sqrt{g y_1}} = \frac{q}{\sqrt{g y_1^3}}$$

Knowing q and y_1, F_1 can be determined. Graphs are available between $\left(\frac{x}{y_1}\right)$ *Vs* $\left(\frac{y}{y_1}\right)$ for different values of Froude No. F_1, as shown in Plate 10.3 (a). Taking different points beyond P along the profile, different values of x and hence that of x/y, can be tabulated. Corresponding to these values of x/y_1 for a fixed F_1, different values of y/y_1, can be read out from Plate 10.3 (a). Hence, different values of x and y are known, where (x, y) is any pt. in the direction of flow w.r.t. the point P (i.e. glacis level at the point of jump formation) as origin, as shown in Fig. 10.5. Hence, the water surface profile after the jump point can be easily plotted.

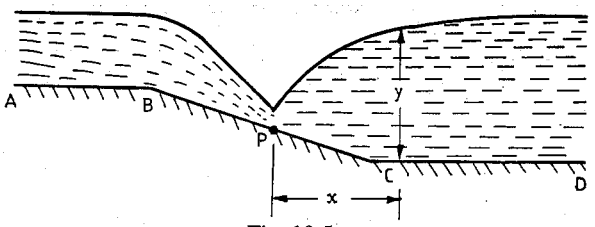


Fig. 10.5

The water surface profile after the jump can also be plotted with the help of curves shown in Plate 10.3 (*b*). Values of $\frac{y}{y_2 - y_1}$ for known values of $\frac{x}{y_2 - y_1}$ can be read out for a given value of F_1 ; where (x, y) in this case are the ordinates of any point on the profile w.r.t. the point of jump formation (P') as origin, as shown in Fig. 10.6. Hence, for any assumed values of x, corresponding values of y can be worked out and profile plotted easily.

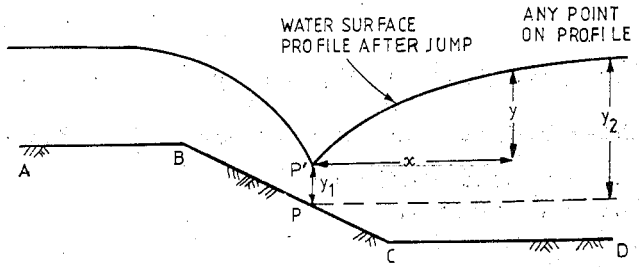


Fig. 10.6

10.5. Hydraulic Jump on a Sloping Glacis as Energy Dissipator

Much has already been said about the use of hydraulic jump phenomenon as energy dissipation device in the design of hydraulic and irrigation structures. The use of 'sloping glacis' for bringing out hydraulic jump to occur is of utmost importance because of the fact : *that the position of the hydraulic jump on a sloping glacis is definite and can be predicted, while on a level floor the position of the jump is unstable. However, on a 'sloping glacis', the energy dissipation is less efficient because* of the vertical component of the velocity remaining intact. In a jump on a glacis, it is only the horizontal component of velocity which takes part in the impact and vertical component remains uneffected.

The length of the jump, *i.e.* the region in which heavy turbulence is created is generally found to be five times the height. Moreover, the start point of the jump is fairly definite but the lower end is indefinite. This point is, however, taken as the place where water surface becomes sensibly level.

Due to heavy turbulence created in the region of the jump. It is necessary to provide a pucca hard floor in this region. A jump if allowed to form on a level surface, cannot be confined precisely to this definite region, because the position of the jump varies through a wide range with a slight change in the discharge. Hence the jump, if formed on a level surface, may not confine itself to the pucca platform and may travel to downstream protection or natural erodible bed of the channel causing deep scours and sometimes even failure of the structure. *Hence, a sloping glacis is always preferred to a horizontal bed for affecting hydraulic jump phenomenon, because the position of the jump on a 'glacis' is always definite although less efficient.* Hence, a sloping glacis having a slope of 2 : 1 to 5 : 1 is generally provided, and hydraulic jump is made to occur on the glacis itself, and in no case lower than the toe of the glacis.

PROBLEMS

1. (*a*) What is meant by 'Hydraulic jump' and how does it helps in designing irrigation structures?

(*b*) Differentiate between 'Sequent depth' and 'Alternate depth'.

(*c*) Derive an expression for expressing energy dissipation obtained by a jump, in terms of initial depth (y_1) and sequent depth (y_2).

2. (*a*) What is a hydraulic jump ? How does it help in dissipating the energy of the water falling over a weir or a dam. What would happen if this energy is not properly dissipated ?

(*b*) What is the importance of "Incoming Froude number", and how does it help in indicating the success of a jump formation ?

(*c*) How would you fix the jump portion when water is flowing over a sloping glacis. Also explain as to how the pre-jump as well as post jump profile can be plotted and what useful purpose will be served by such plottings ?

PLATE NO. 10.2
ENERGY OF FLOW CURVES
FOR DIFFERENT VALUESOF q in
CUMECS/METRE
DEPTH (y) – (m)
ENERGY OF FLOW $E_f = y + \frac{y^2}{2g}$ (m)
LOCUS OF CRITICAL DEPTH
DEPTH LINE

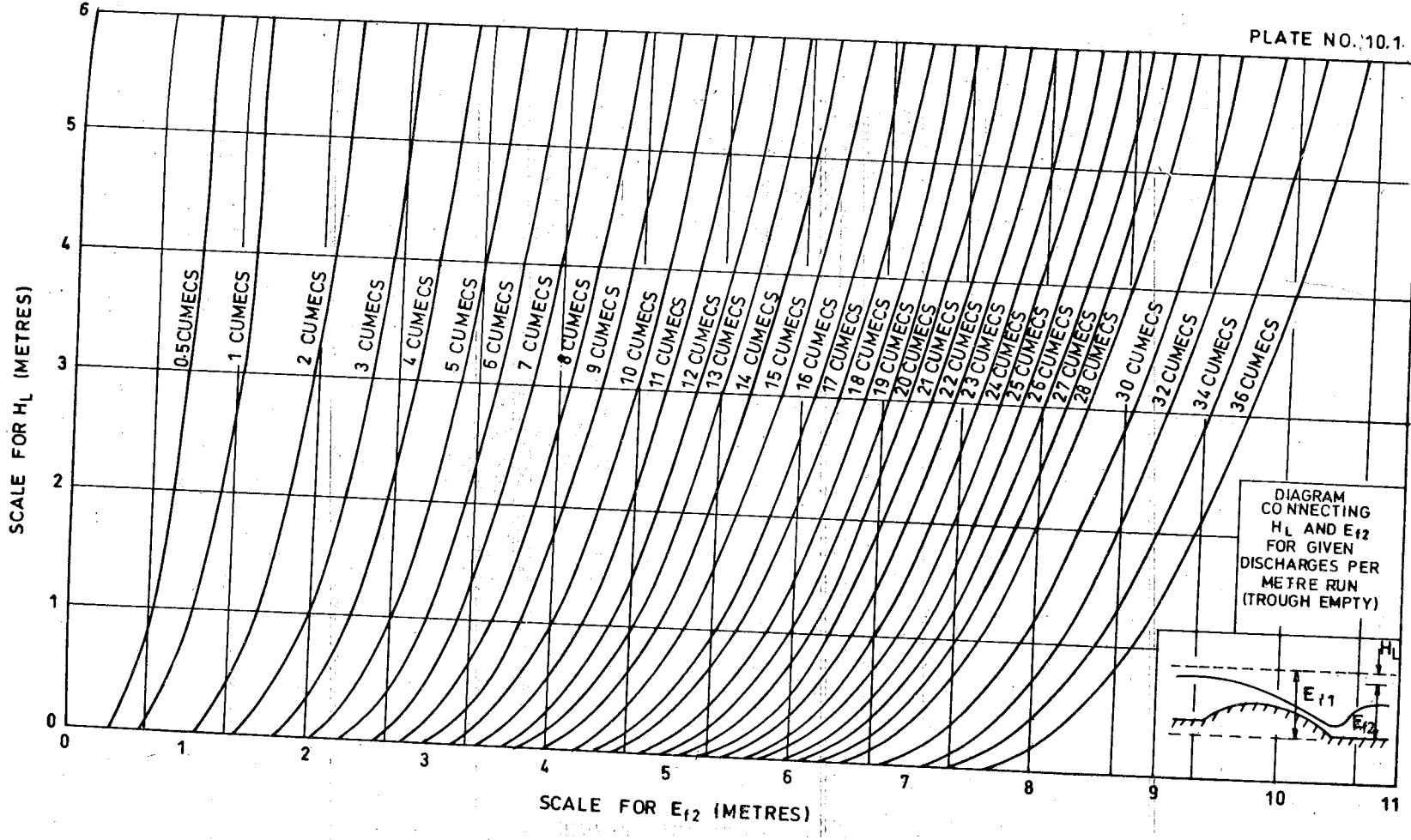
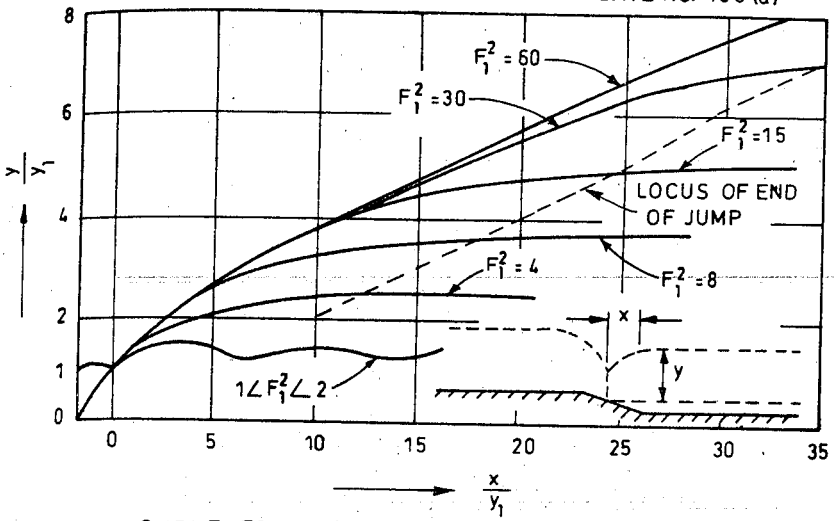
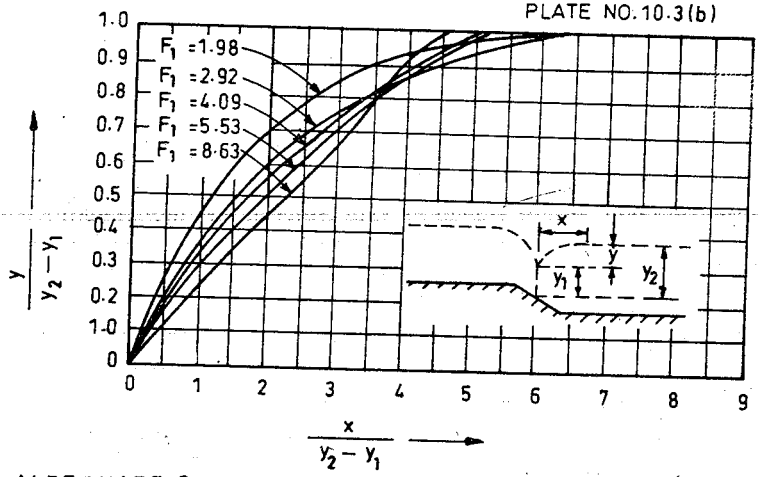

PLATE NO. 10.1
SCALE FOR H_L (METRES)
SCALE FOR E_{f2} (METRES)
0.5 CUMECS
1 CUMECS
2 CUMECS
3 CUMECS
4 CUMECS
5 CUMECS
6 CUMECS
7 CUMECS
8 CUMECS
9 CUMECS
10 CUMECS
11 CUMECS
12 CUMECS
13 CUMECS
14 CUMECS
15 CUMECS
16 CUMECS
17 CUMECS
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22 CUMECS
23 CUMECS
24 CUMECS
25 CUMECS
26 CUMECS
27 CUMECS
28 CUMECS
30 CUMECS
32 CUMECS
34 CUMECS
36 CUMECS
DIAGRAM CONNECTING H_L AND E_{f2} FOR GIVEN DISCHARGES PER METRE RUN (TROUGH EMPTY)
H_L
E_{f1}
E_{f2}

PLATE NO. 10.3 (a)



CURVE FOR PLOTTING POST JUMP PROFILE

PLATE NO. 10.3 (b)



ALTERNATE CURVE FOR PLOTTING POST JUMP PROFILE

11

Theories of Seepage and Design of Weirs and Barrages

11.1. Failure of Hydraulic Structures Founded on Pervious Foundations

Hydraulic structures such as dams, weirs, barrages, head regulators, cross-drainage works, etc. may either be founded on an impervious solid rock foundation or on a pervious foundation. Whenever, such a structure is founded on a pervious foundation, it is subjected to seepage of water beneath the structure, in addition to all other forces to which it will be subjected when founded on an impervious rock foundation. In India, most of these hydraulic structures are required to be founded on alluvial soil foundations, which do allow seepage beneath them. The water seeping below the body of the hydraulic structure, endangers the stability of the structure and may cause its failure, either by :

(*i*) Piping ; or

(*ii*) by Direct uplift.

(*i*) **Failure by Piping or Undermining.** When the seepage water retains sufficient residual force at the emerging downstream end of the work, it may lift up the soil particles. This leads to increased porosity of the soil by progressive removal of soil from beneath the foundation. The structure may ultimately subside into the hollow so formed, resulting in the failure of the structure.

(*ii*) **Failure by Direct Uplift.** The water seeping below the structure, exerts an uplift pressure on the floor of the structure. If this pressure is not counterbalanced by the weight of the concrete or masonry floor, the structure will fail by a rupture of a part of the floor.

The above concepts of the failure of hydraulic structures due to sub-surface flow were introduced by Bligh, on the basis of experiments and the research work conducted after the failure of Khanki weir, which was designed on experience and intuition without any rational theory.

11.2. Bligh's Creep Theory for Seepage Flow

According to Bligh's Theory, the percolating water follows the outline of the base of the foundation of the hydraulic structure. In other words, water creeps along the bottom contour of the structure. The length of the path thus traversed by water is called the *length of the creep*. Further, it is assumed in this theory, that the loss of head is proportional to the length of the creep. If H_L is the total head loss between the upstream and the downstream, and L is the length of creep, then the loss of head per unit of creep

length (*i.e.* H_L/L) is called the *hydraulic gradient.* Further, Bligh makes no distinction between horizontal and vertical creep.

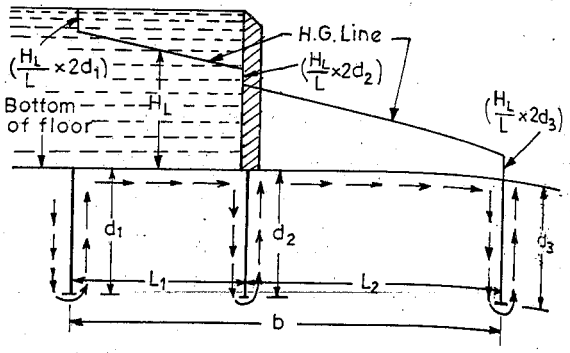


Fig. 11.1. Bligh's Creep.

Consider a section as shown in Fig. 11.1. Let H_L be the difference of water levels between upstream and downstream ends. (No water is shown on d/s side in Fig. 11.1). Water will seep along the bottom contour as shown by arrows. It starts percolating at *A* and emerges at *B*. The total length of creep is given by

$$\begin{aligned} L &= d_1 + d_1 + L_1 + d_2 + d_2 + L_2 + d_3 + d_3 \\ &= 2d_1 + (L_1 + L_2) + 2d_2 + 2d_3 \\ &= (L_1 + L_2) + 2\,[d_1 + d_2 + d_3] \\ &= b + 2\,(d_1 + d_2 + d_3) \end{aligned}$$

Head loss per unit length or hydraulic gradient

$$= \left[\frac{H_L}{b + 2.\,(d_1 + d_2 + d_3)}\right] = \frac{H_L}{L}$$

Head losses equal to $\left(\frac{H_L}{L}\times 2d_1\right), \left(\frac{H_L}{L}\times 2d_2\right)\left(\frac{H_L}{L}\times 2d_3\right)$; will occur respectively, in the planes of three vertical cut offs. The hydraulic gradient line (H.G. Line) can then be drawn as shown in Fig. 11.1.

(*i*) **Safety Against Piping or Undermining.** According to Bligh, the safety against piping can be ensured by providing sufficient creep length, given by $L = C.H_L$, where *C* is Bligh's coefficient for the soil. Different values of *C* for different types of soils are tabulated in Table 11.1.

Table 11.1. Values of Bligh's Safe Hydraulic Gradient for different types of Soils

S.No.	*Type of soil*	*Value of C*	*Safe Hydraulic gradient should be less than*
1.	Fine micaceous sand (as in North Indian Rivers)	15	1/15
2.	Coarse grained sand (as in Central and South Indian Rivers)	12	1/12
3.	Sand mixed with boulder and gravel, and for loam soil	5 to 9	1/5 to 1/9
4.	Light sand and mud	8	1/8

Note : The hydraulic gradient, *i.e.* H_L/L is then equal to $1/C$. Hence, it may be stated that the hydraulic gradient must be kept under a safe limit in order to ensure safety against piping.

(*ii*) **Safety against uplift pressure.** The ordinates of the H.G. line above the bottom of the floor represent the residual uplift water head at each point. Say for example, if at

any point, the ordinate of H.G. line above the bottom of the floor is 1 m, then 1 m head of water will act as uplift at that point. If h' metres is this ordinate, then water pressure equal to h' metres will act at this point, and has to be counterbalanced by the weight of the floor of thickness say t.

$\therefore$ Uplift pressure $= \gamma_w h'$

where γ_w is the unit wt. of water

Downward pressure $= (\gamma_w . G.)\, t$

where G is the specific gravity of the floor material.

For equilibrium

$$\gamma_w \cdot h' = \gamma_w . G . t$$

$$h' = Gt$$

Subtracting t on both sides, we get

$$(h' - t) = (G.t - t) = t\,(G - 1)$$

or

$$\boxed{t = \left(\frac{h' - t}{G - 1}\right) = \left(\frac{h}{G - 1}\right)} \qquad ...(11.1)$$

where $(h' - t) = h$ is the ordinate of the H.G. line above the top of the floor. $(G - 1)$ is the submerged specific gravity of the floor material. For concrete, G may be taken equal to 2.4. Hence, the thickness of the floor can be easily determined by using the equation (11.1).This is generally increased by 33%, so as to allow a suitable factor of safety.

It may be mentioned that the floor thickness has to be designed according to equation (11.1) only for the downstream floor and for the worst conditions *i.e.* when maximum ordinates of H.G. line occur. The water standing on the upstream floor, more than counterbalances the uplift caused by the same water, and hence, *only a nominal floor thickness is required on the upstream side*, so as to resist wear, impact of flowing water, etc.

Hence, while designing aprons of hydraulic structures on Bligh's theory for sub-surface flow, the floor thickness, is designed in accordance with the above rules, and sufficient length of pucca floor given by $L = C.H_L$ is provided, so as to ensure a safe value of hydraulic gradient. This will be discussed in details in article 11.5.

11.3. Lane's Weighted Creep Theory

Bligh, in his theory, had calculated the length of the creep, by simply adding the horizontal creep length and the vertical creep length, thereby making no distinction between the two creeps. However, Lane, on the basis of his analysis carried out on about 200 dams all over the world, stipulated that the horizontal creep is less effective in reducing uplift (or in causing loss of head) than the vertical creep. He, therefore, suggested a weightage factor of $\frac{1}{3}$ for the horizontal creep, as against 1.0 for the vertical creep.

Thus in Fig. 11.1, the total Lane's creep length (L_l) is given by

$$L_l = (d_1 + d_1) + \frac{1}{3} L_1 + (d_2 + d_2) + \frac{1}{3} L_2 + (d_3 + d_3)$$

$$= \frac{1}{3}.(L_1 + L_2) + 2(d_1 + d_2 + d_3) = \frac{1}{3}.b + 2(d_2 + d_2 + d_3)$$

To ensure safety against piping, according to this theory, the creep length L_l must not be less than $C_1 H_L$, where H_L is the head causing flow, and C_1 is Lane's creep coefficient given in table 11.2.

Table 11.2. Values of Lane's Safe Hydraulic Gradient for different types of Soils

S.No.	*Type of soil*	*Value of Lane's Coefficient, C_1*	*Safe Lane's Hydraulic gradient should be less than*
1.	Very find sand or silt	8.5	1/8.5
2.	Fine sand	7.0	1/7
3.	Coarse sand	5.0	1/5
4.	Gravel and sand	3.5 to 3.0	1/3.5 to 1/3
5.	Boulders, gravels and sand	2.5 to 3.0	1/2.5 to 1/3
6.	Clayey soils	3.0 to 1.6	1/3 to 1/1.6

Lane's theory was an improvement over Bligh's theory, but however, was purely empirical without any rational basis, and hence, is generally not adopted in any designs. Bligh's theory, though is still used (even after the invention of modern Khosla's theory), but Lane's theory is practically nowhere used, and is having only a theoretical importance.

11.4. Khosla's Theory and Concept of Flow Nets

Many of the important hydraulic structures, such as weirs and barrages, were designed on the basis of Bligh's theory between the period 1910 to 1925. In 1926-27, the upper Chenab canal syphons, designed on Bligh's theory, started posing undermining troubles. Investigations started, which ultimately lead to Khosla's theory.

A detailed description of this theory is available in C.B.I. publication No. 12, which is available at Publication's Division at Civil Lines, Delhi. The main principles of this theory are summarised below :

(1) The seeping water does not creep along the bottom contour of pucca floor as stated by Bligh, but on the other hand, this water moves along a set of stream-lines as shown in Fig. 11.3. This steady seepage in a vertical plane for a homogeneous soil can be expressed by Laplacian equation.

$$\frac{d^2\phi}{dx^2} + \frac{d^2\phi}{dz^2} = 0$$

where ϕ = Flow potential = Kh where K is the coefficient of permeability of soil as defined by Darcy's law, and h is the residual head at any point within the soil.

The above equation represents two sets of curves intersecting each other orthogonally (Fig. 11.2). One set of lines is called *Streamlines*, and the other set is called *Equipotential lines*. The resultant flow diagram showing both the sets of curves is called a *Flow Net.*

Stream Lines. The streamlines represent the paths along which the water flows through the sub-soil. Every particle entering the soil at a given point upstream of the work, will trace out its own path and will represent a streamline. The first streamline follows the bottom contour of the works and is the same as Bligh's path of creep. The remaining streamlines follow smooth curves transiting slowly from the outline of the foundation to a semi-ellipse, as shown in Fig. 11.2.

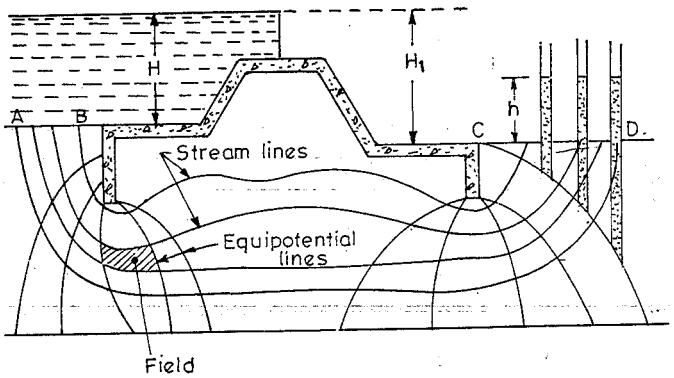


Fig. 11.2. Khosla's Flow Net.

Equipotential Lines. (1) Treating the downstream bed as datum and assuming no water on the downstream side, it can be easily stated that every streamline possesses a head equal to h_1 while entering the soil; and when it emerges at the down-stream end into the atmosphere, its head is zero. Thus, the head h_1 is entirely lost during the passage of water along the streamline.

Further, at every intermediate point in its path, there is certain residual head (h) still to be dissipated in the remaining length to be traversed to the downstream end. This fact is applicable to every streamline, and hence, there will be points on different streamlines having the same value of residual head h. If such points are joined together, the curve obtained is called an *equipotential line.*

Every water particle on line AB is having a residual head $h = h_1$ and on CD is having a residual head $h = 0$, and hence, AB and CD are equipotential lines.

Since an equipotential line represents the joining of points of equal residual head, hence if piezometers were installed on an equipotential line, the water will rise in all of them up to the same level as shown in Fig. 11.2.

(2) The seepage water exerts a force at each point in the direction of flow and tangential to the streamlines as shown in Fig. 11.3. This force (F) has an upward

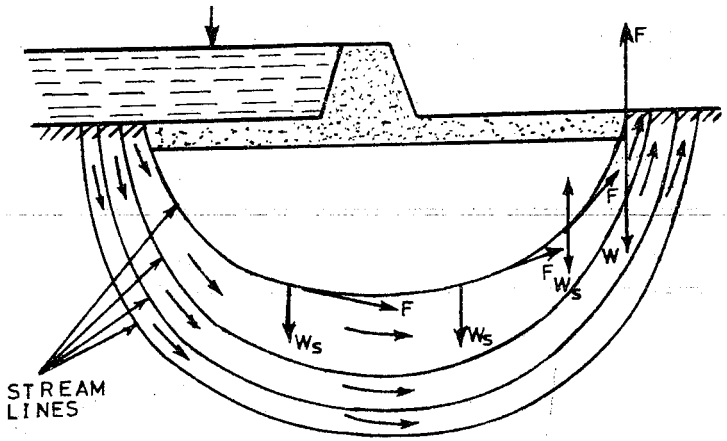


Fig. 11.3

component from the point where the streamline turns upward. For soil grains to remain stable, the upward component of this force should be counterbalanced by the submerged weight of the soil grain. This force has the maximum disturbing tendency at the exit end, because the direction of this force at the exit point is vertically upward, and hence full force acts as its upward component. For the soil grain to remain stable, the submerged weight of soil grain should be more than this upward disturbing force. The disturbing force at any point is proportional to the gradient of pressure of water at that point (*i.e.* dp/dt). This gradient of pressure of water at the exit end, is called the **exit gradient**. In order that the soil particles at exit remain stable, the upward pressure at exit should be safe. In other words, the exit gradient should be safe.

Critical Exit Gradient. This exit gradient is said to be critical, when the upward disturbing force on the grain is just equal to the submerged weight of the grain at the exit. When a factor of safety equal to 4 or 5 is used, the exit gradient can then be taken as safe. In other words, an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ of the critical exit gradient is ensured, so as to keep the structure safe against piping.

The submerged weight (W_s) of a unit volume of soil is given as :

$$\gamma_w (1 - n) (S_s - 1)$$

where γ_w = unit weight of water.

S_s = sp. gravity of soil particles

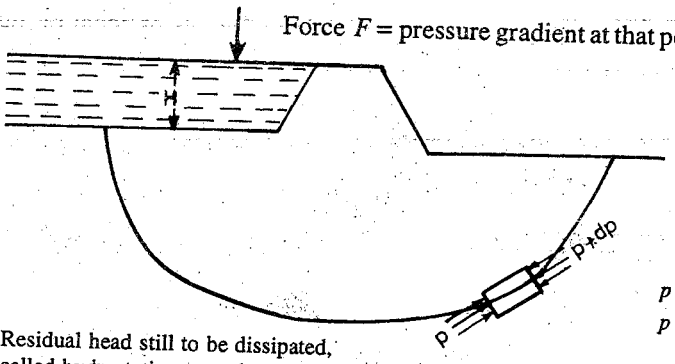
n = porosity of the soil material.

For critical conditions to occur at the exit point,

$$F = W_s.$$

where F is the upward disturbing force on the grain

Force F = pressure gradient at that point $= \frac{dp}{dl} = \gamma_w \cdot \frac{dh}{dl}$



$p = \gamma_w \cdot h$

$p + dp = \gamma_w (h + dh)$

where, h = Residual head still to be dissipated, called hydrostatic excess head.

Fig. 11.4

$$\therefore \quad \gamma_w \cdot \frac{dh}{dl} = \gamma_w (1 - n) (S_s - 1)$$

or

$$\frac{dh}{dl} = (1 - n) (S_s - 1) \qquad ...(11.2)$$

where $\frac{dh}{dl}$ represents the rate of loss of head or the gradient at the exit end.

Under critical conditions, the critical exit gradient is equal to $(1-n)(S_s-1)$. For most of the river sands, $S_s \cong 2.65$ and $n \cong 0.4$, then the value of critical exit gradient $=(1-0.4)(2.65-1)$

$$= 0.6 \times 1.65 = 0.99 \cong \mathbf{1.0}$$

Hence, an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ of the critical gradient means that an exit gradient equal to $\frac{1}{4}$ to $\frac{1}{5}$ has to be provided for keeping the structure safe against piping.

Values of safe exit gradient for some of the subsoils are given in Table 11.3.

Table 11.3. Values of Khosla's Safe Exit Gradient for different types of Soils

Type of soil	*Khosla's Safe Exit Gradient*
Shingle	0.25 to 0.20
Course sand	0.20 to 0.17
Fine sand	0.17 to 0.14

Khosla's theory of flow nets made it very clear that the loss of head does not take place uniformly, in direct proportion to the creep length, as stated by Bligh. In fact, it depends upon the whole geometry of the figure, *i.e.* the shape of foundation, depth of impervious boundary and levels of u/s and d/s beds. When the *equipotential lines are traced to be closer, the rate of loss of head will definitely be quicker and vice versa.*

It can, hence be concluded that the safety against piping can not be obtained by providing sufficient floor length, as stated by Bligh, but can be obtained by keeping the exit gradient well below the critical value. The exit gradient may not be safe even if the average hydraulic gradient of Bligh $\left(i.e.\ \frac{l}{C}\right)$ is safe.

(3) Undermining of the floor starts from the downstream end of the d/s pucca floor, and if not checked, it travels upstream towards the weir wall. The undermining starts only when the exit gradient is unsafe for the subsoil on which the weir is founded. *It is, therefore, absolutely necessary to have a reasonably deep vertical cut-off at the downstream end of the d/s pucca floor to prevent undermining.* The depth of this d/s vertical cut off is governed by two considerations *i.e.*

(*i*) maximum depth of scour ; (*ii*) safe exit gradient.

While designing a weir, downstream cutoff from the maximum scoured depth considerations is, first of all, provided, and checked for exit gradient. If a safe value of exit gradient is not obtained, then the depth of cutoff is increased. The depth of cutoff is also governed and limited by practical considerations, as the execution of very deep cutoff may be difficult or unpracticable at site.

A weir or a barrage may fail not only due to seepage (*i.e.* sub-surface flow) as stated by Bligh, but may also fail due to the surface flow. The surface flow (*i.e.* when flood water flows over the weir crest) may cause scour, dynamic action ; and in addition, will

cause uplift pressures in the jump trough* (if the hydraulic jump forms on the downstream). These uplift pressures must be investigated for various flow conditions. The maximum uplift due to this dynamic action (*i.e.* for surface flow) should then be compared with the maximum uplift under steady seepage (*i.e.* for sub-surface flow) ; and the maximum of the two chosen for designing the aprons and the floors of the weirs. All these modern aspects and other details about designing weirs on permeable foundations, as per the Khosla's theory, have been discussed in article 11.6.

Khosla's theory differs from Bligh's theory in all the above respects, but owing to the simplicity, Bligh's theory is still used for design of small works. A minimum practical thickness for the floor and a deep vertical cutoff at the downstream end is, however, always provided, in addition to the requirements of Bligh's theory. However, on major works, Bligh's theory should never be used, as it would lead to expensive and unsafe erroneous designs.

11.4.1. Khosla's method of independent variables for determination of pressures and exit gradient for seepage below a weir or a barrage. In order to know as to how the seepage below the foundation of a hydraulic structure is taking place, it is necessary to plot the flownet. In other words, we must solve the Laplacian equations. This can be accomplished either by mathematical solution of the Laplacian equations, or by Electrical analogy method, or by graphical sketching by adjusting the streamlines and equipotential lines w.r.t. the boundary conditions. These are complicated methods and are time consuming. Therefore, for designing hydraulic structures such as weirs or barrages on pervious foundations, Khosla has evolved a simple, quick and an accurate approach, called *Method of Independent Variables.*

In this method, a complex profile like that of a weir is broken into a number of simple profiles, each of which can be solved mathematically. Mathematical solutions of flownets for these simple standard profiles have been presented in the form of equations given in Fig. 11.5, and curves given in Plate 11.1, which can be used for determining the percentage pressures at the various key points. The simple profiles which are most useful are :

(*i*) A straight horizontal floor of negligible thickness with a sheet pile line on the u/s end and d/s end [Fig. 11.5 (*a*) and (*b*)].

(*ii*) A straight horizontal floor depressed below the bed but without any vertical cut-offs [Fig. 11.5 (*c*)].

(*iii*) A straight horizontal floor of negligible thickness with a sheet pile line at some intermediate point [Fig. 11.5 (*d*)].

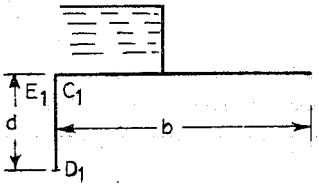
The key points are the junctions of the floor and the pile lines on either side, and the bottom point of the pile line, and the bottom corners in the case of a depressed floor. The percentage pressures at these key points for the simple forms into which the complex profile has been broken is valid for the complex profile itself, if corrected for

(*a*) correction for the mutual interference of piles ;

(*b*) correction for thickness of floor ;

(*c*) correction for the slope of the floor.

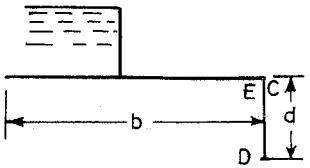
* For detailed description, please refer to Article 11.6.11.



$$\phi_{C_1} = 100 - \phi_E$$

$$\phi_{D_1} = 100 - \phi_D$$

(*a*)



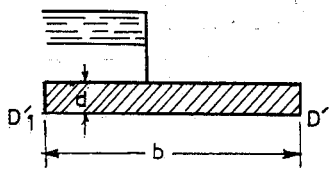
$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 1}{\lambda}\right)$$

where $\lambda = \dfrac{1 + \sqrt{1 + \alpha^2}}{2}$

$\alpha = \dfrac{b}{d}$ (respective)

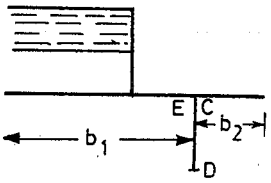
(*b*)



$$\phi_{D'} = \frac{2}{3}(\phi_E - \phi_D) + \frac{3}{\alpha^2}$$

$$\phi'_{D_1} = 100 - \phi_{D'}$$

(*c*)



$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$\phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1 + \alpha_1^2} + \sqrt{1 + \alpha_2^2}}{2}$

$\lambda_1 = \dfrac{\sqrt{1 + \alpha_1^2} - \sqrt{1 + \alpha_2^2}}{2}$

$\alpha_1 = b_1/d$

$\alpha_2 = b_2/d$

(*d*)

Fig. 11.5. Khosla's simple profiles for a weir of complex profile.

These corrections are described below :

(*a*) **Correction for the Mutual Interference of Piles.** The correction C to be applied as percentage of head due to this effect, is given by

$$C = 19\sqrt{\frac{D}{b'}}\left[\frac{d + D}{b}\right] \qquad ...(11.3)$$

where b' = The distance between two pile lines.

D = The depth of the pile line, the influence of which has to be determined on the neighbouring pile of depth d. D is to be measured *below the level at which interference is desired.*

d = The depth of the pile on which the effect is considered.

b = Total floor length.

This correction *is positive for the points in the rear or back water; and substractive for the points forward in the direction of flow. This equation does not apply to the effect of an outer pile on an intermediate pile, if the intermediate pile is equal to or smaller than the outer pile and is at a distance less than twice the length of the outer pile.*

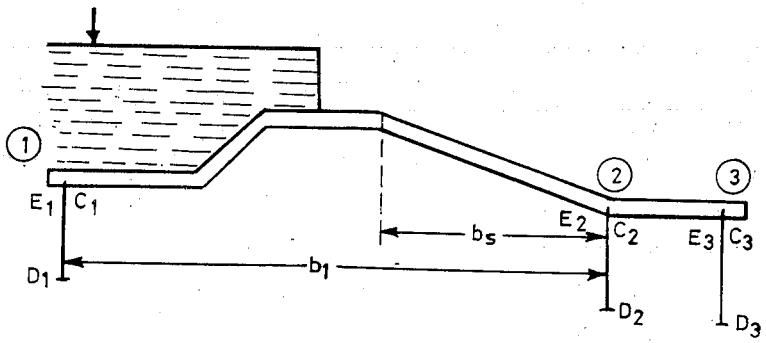


Fig. 11.6

Suppose in the above Fig. 11.6, we are considering the influence of the pile No. (2) on pile No. (1) for correcting the pressure at C_1. Since the point C_1 is in the rear, this correction shall be + ve. While the correction to be applied to E_2 due to pile No. (1) shall be negative, since the point E_2 is in the forward direction of flow. Similarly, the correction at C_2 due to pile No. (3) is positive, and the correction at E_2 due to pile No. (2) is negative.

(*b*) **Correction for the Thickness of Floor.** In the standard form profiles, the floor is assumed to have negligible thickness. Hence, the percentage pressures calculated by Khosla's equations or graphs shall pertain to the top levels of the floor. While the actual junction points E and C are at the bottom of the floor. Hence, the pressure at the actual points are calculated by assuming a straight line pressure variation. Since the corrected pressure at E_1 should be less than the calculated pressure at E_1', the correction to be applied for the point E_1 shall be – ve. Similarly, the pressure calculated C_1' is less than the corrected pressure at C_1, and hence, the correction to be applied at point C_1 is + ve.

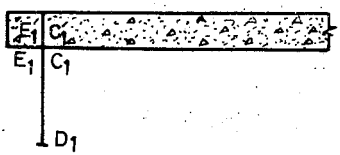


Fig. 11.7

(*c*) **Correction for the Slope of the Floor.** A correction is applied for a sloping floor, and is taken as + ve for the down, and – ve for the *up* slopes following the direction

of flow. Values of correction of standard slopes such as 1 : 1, 2 : 1, 3 : 1, etc. are tabulated in Table 11.4.

Table 11.4

Slope Horizontal : Vertical	*Correction factor*
1 : 1	11.2
2 : 1	6.5
3 : 1	4.5
4 : 1	3.3
5 : 1	2.8
6 : 1	2.5
7 : 1	2.3
8 : 1	2.0

The correction factor given above is to be multiplied by the horizontal length of the slope and divided by the distance between the two pile lines between which the sloping floor is located. This correction is applicable only to the key points of the pile line fixed at the start or the end of the slope.

Thus, in Fig. 11.6, this correction is applicable only to point E_2. Since the slope is down at point E_2 in the direction of flow, hence, the correction shall be + ve and will be equal to the correction factor for this slope (Table 11.4) multiplied by b_s/b_1, where b_s and b_1 are shown in Fig. 11.6.

Exit Gradient (G_E). It has been determined that for a standard form consisting of a floor length b with a vertical cutoff of depth d, the exit gradient at its downstream end is given by

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} \qquad \text{...(11.4)}$$

where $$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

and $$\alpha = \frac{b}{d}.$$

From the curve of Plate 11.2 ; for any value of α, *i.e.* $\frac{b}{d}$, the corresponding value of $\frac{1}{\pi\sqrt{\lambda}}$ can be read. Knowing H and d, the value of G_E can be easily calculated. The exit gradient so calculated must lie within safe limits as given in Table 11.5.

Table 11.5

Type of soil	*Safe exit gradient*
Shingle	$\frac{1}{4}$ to $\frac{1}{5}$ (0.25 to 0.20)
Coarse Sand	$\frac{1}{5}$ to $\frac{1}{6}$ (0.20 to 0.17)
Fine Sand	$\frac{1}{6}$ to $\frac{1}{7}$ (0.17 to 0.14)

The uplift pressures must be kept as low as possible consistent with the safety at the exit, so as to keep the floor thickness to the minimum.

It is obvious from equation (11.4), that if $d = 0$; G_E is infinite. *Hence, it becomes essential that a vertical cutoff at the downstream end must be provided.*

Example 11.1. *Determine the percentage pressures at various key points in Fig. 11.8. Also determine the exit gradient and plot the hydraulic gradient line for pond level on u/s and no flow on d/s.*

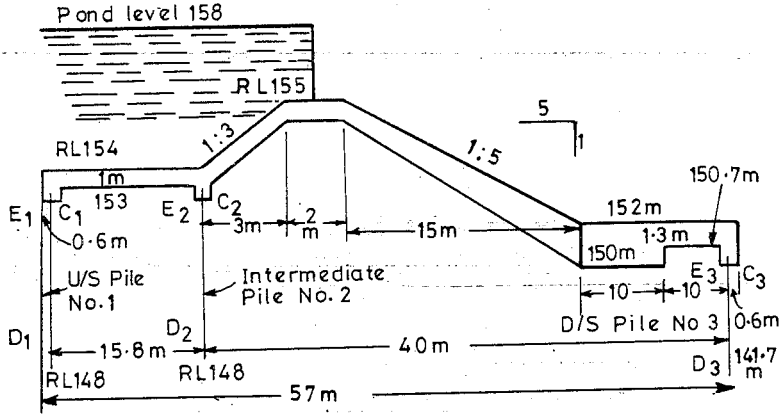


Fig. 11.8

Solution.

(1) **For Upstream Pile Line No. (1)**

Total length of the floor $= b = 57.0$ m.

Depth of u/s pile line $= d = 154.00 - 148.00 = 6.0$ m

$$\alpha = \frac{b}{d} = \frac{57.0}{6.0} = 9.5$$

$$\frac{1}{\alpha} = \frac{1}{9.5} = 0.105$$

From curve Plate 11.1 (*a*)

$$\phi_{C_1} = 100 - 29 = 71\%$$

$$\phi_{D_1} = 100 - 20 = 80\%$$

These values of ϕ_{C_1} and ϕ_{D_1} must be corrected for three corrections as below :

Corrections for ϕ_{C_1}

(*a*) *Correction at C_1 for Mutual Interference of Piles.* ϕ_{C_1} is affected by intermediate pile No. 2.

$$\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right) \qquad ...(11.3)$$

where D = Depth of pile No. 2.
$= 153.00 - 148.00 = 5.0$ m
d = Depth of pile No. 1 $= 153.0 - 148.00 = 5.0$ m
b' = Distance between two piles $= 15.8$ m.
b = Total floor length $= 57.0$ m.

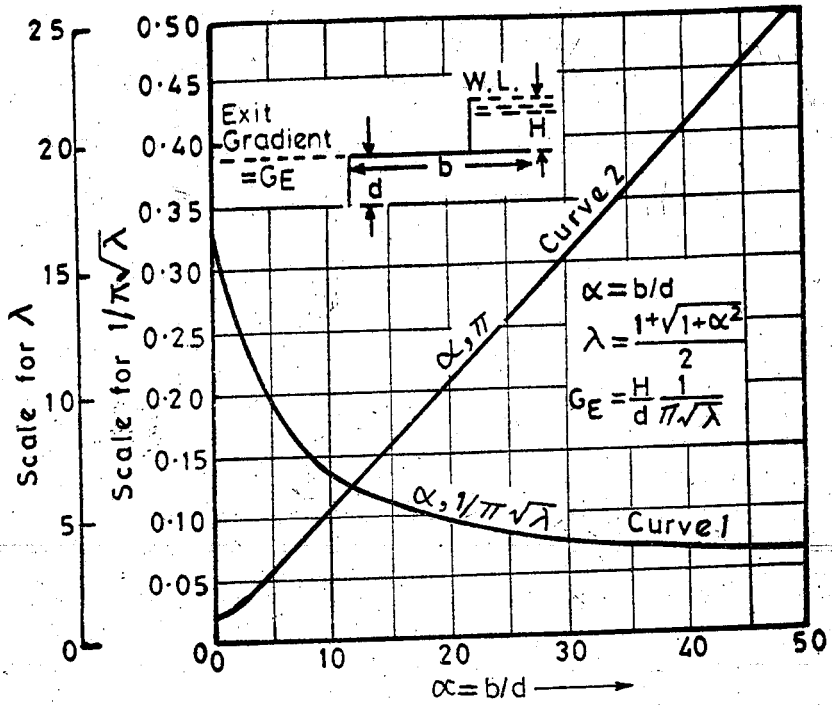

Scale for λ
Scale for 1/π√λ
Exit Gradient =GE
W.L.
H
b
d
Curve 2
α, π
α = b/d
λ = (1+√(1+α²))/2
GE = H/d · 1/π√λ
α, 1/π√λ
Curve 1
α = b/d

Plate 11.2

$$\text{Correction} = 19\sqrt{\frac{5}{15.7}}\left[\frac{5+5}{57}\right] = \mathbf{1.88\%}$$

Since the point C_1 is in the rear in the direction of flow, the correction is + ve.

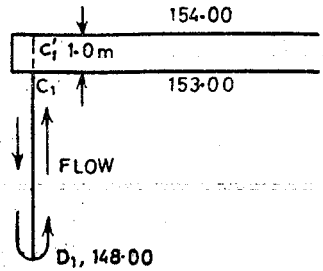


Fig. 11.9

$\therefore$ Correction due to pile interference on C_1

$$= 1.88\% \text{ (+ ve)} \qquad ...(i)$$

(*b*) *Correction at C_1 due to thickness of floor.*

Pressure calculated from curve is at C_1', (Fig. 11.9) but we want the pressure at C_1. Pressure at C_1 shall be more than at C_1' as the direction of flow is from C_1 to C_1' as shown; and hence, the correction will be + ve and

$$= \left[\frac{80\% - 71\%}{154.0 - 148.0}\right] \times (154.00 - 153.00) = \frac{9}{6} \times 1 = 1.5\% \text{ (+ ve)} \qquad ...(ii)$$

(*c*) *Correction due to slope at C_1 is nil, as this point is neither situated* at the start nor at the end of a slope.

$\therefore$ Corrected $\phi_{C_1} = 71\% + 1.88\% + 1.5\% = 74.38\%$

Hence **corrected** $\phi_{C_1} = \mathbf{74.38\%}$ **Ans.**

and $\phi_{D_1} = 80\%$

(2) For Intermediate Pile Line No. (2)

$$d = 154.00 - 148.00 = 6.0 \text{ m}$$

$$b = 57.0 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57.0}{6.0} = 9.5$$

Using curves of Plate 11.1 (*b*), we have b_1 in this case

$$= 0.6 + 15.8 = 16.4 \text{ m}$$

and $b = 57.0$ m

$\therefore$ $\dfrac{b_1}{b} = \dfrac{16.4}{57.0} = 0.298.$

$$1 - \frac{b_1}{b} = 1 - 0.298 = 0.702.$$

$$\phi_{E_2} = 100 - 30\% = 70\%$$

(where 30% is ϕ_C for a base ratio of 0.702 and $\alpha = 9.5$)

$\phi_{C_2} = 56\%$ (for a base ratio 0.298 and $\alpha = 9.5$)

$\phi_{D_2} = 100 - 37 = 63\%$

(where 37% is ϕ_D for a base ratio of 0.702 and $\alpha = 9.5$)

Corrections for ϕ_{E_2}

(*a*) *Correction at E_2 for sheet pile lines.* Pile No. (1) will affect the pressure at E_2 and since E_2 is in the forward direction of flow, this correction shall be – ve. The amount of this correction is given as :

$$19\sqrt{\frac{D}{b'}}\cdot\left[\frac{d+D}{b}\right]$$

where D = Depth of pile No. 1, the effect of which is considered = 153.0 – 148.0 = 5.0 m

d = depth of pile No. 2, the effect on which is considered = 153.0 – 148.0 = 5.0 m.

b' = Distance between the two piles = 15.8 m.

b = Total floor length = 57.0 m.

$$\text{Correction} = 19\sqrt{\frac{5}{15.7}}\left(\frac{5+5}{57}\right) = \mathbf{1.88\%}\ (-\text{ve})$$

(*b*) *Correction at E_2 due to floor thickness*

$$= \frac{\text{Obs. } \phi_{E_2} - \text{Obs. } \phi_{D_2}}{\text{Distance between } E_2D_2} \times \text{Thickness of floor}$$

$$= \left(\frac{70\% - 63\%}{154.0 - 148.0}\right) \times 1.0 = \frac{7}{6} \times 1.0 = 1.17\%.$$

Since the pressure observed is at E_2' and not at E_2, (Fig. 11.10) and by looking at the direction of flow, it can be stated easily that the pressure at E_2 shall be less than that at E_2', hence, this correction is *negative.*

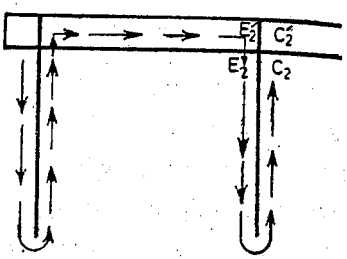


Fig. 11.10

∴ Correction at E_2 due to floor thickness

$$= \mathbf{1.17\%}\ (-\text{ve}).$$

(*c*) *Correction at E_2 due to slope is nil*, as the point E_2 is neither situated at the start of a slope nor at the end of a slope.

Hence, corrected percentage pressure at E_2

$$= \textbf{Corrected } \phi_{E_2} = 70\% - 1.88\% - 1.17\% = \mathbf{66.95\%}\ \textbf{Ans.}$$

Corrections for ϕ_{C_2}

(*a*) *Correction at C_2 due to pile interference.* Pressure at C_2 is affected by pile No. (3) and since the point C_2 is in the back water in the direction of flow, this correction is + ve. The amount of this correction is given as :

$$= 19\sqrt{\frac{D}{b'}} \times \left(\frac{D+d}{b}\right)$$

where D = Depth of pile No. (3), the effect of which is considered below the level at which interference is desired

= 153.0 – 141.7 = 11.3 m

d = Depth of pile No. 2, the effect on which is considered

= 153.0 – 148.0 = 5.0 m.

b' = Distance between the pile 2 and pile 3 = 40.0 m

b = Total floor length = 57.0 m.

$$\text{Correction} = 19\sqrt{\frac{11.0}{40.0}}\left(\frac{11.3+5.0}{57.0}\right) = \mathbf{2.89\%}\ (+\text{ ve})$$

(*b*) *Correction at C_2 due to floor thickness.* From Fig. 11.10, it can be easily stated that the pressure at C_2 shall be more than that at C_2', and since the observed pressure is at C_2', this correction shall be + ve and its amount is the same as was calculated for the point $E_2 = 1.17\%$.

Hence, correction at C_2 due to floor thickness = 1.17% (+ ve).

(*c*) *Correction at C_2 due to slope.* Since the point C_2 is situated at the start of a slope of 3 : 1, *i.e.* an up slope in the direction of flow ; the correction is *negative.*

Correction factor for 3 : 1 slope from Table 11.4 = 4.5

Horizontal length of the slope = 3 m.

Distance between two pile lines between which the sloping floor is located = 40.0 m.

$$\therefore \text{ Actual correction} = 4.5 \times \left(\frac{3}{40}\right) = 0.34\%\ (-\text{ ve})$$

Hence, **corrected** ϕ_{C_2}

$$= 56\% + 2.89\% + 1.17\% - 0.34\% = \mathbf{59.72\%}$$

(3) **Downstream Pile Line**

$$d = 152.0 - 141.7 = 9.3 \text{ m}$$

$$b = 57.0 \text{ m.}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{10.3}{57.0} = 0.181.$$

From curves of Plate 11.1 (*a*), we get

$$\phi_{D_3} = \mathbf{32\%}$$

$$\phi_{E_3} = 38\%$$

Corrections for ϕ_{E_3}

(*a*) *Correction due to piles.* The point E_3 is affected by pile No. 2, and since E_3 is in the forward direction of flow from pile No. 3, this correction is *negative* and its amount is given by

$$19\sqrt{\frac{D}{b}}\left(\frac{d+D}{b}\right)$$

where D = Depth of pile No. 2, the effect of which is considered

= 150.7 – 148.0 = 2.7 m.

d = Depth of pile No. 3, the effect on which is considered

= 150.7 – 141.7 = 9.0 m.

b' = Distance between piles = 40.0 m.

b = Total floor length = 57.0 m.

The correction $= 19\sqrt{\frac{2.7}{40}} \times \left(\frac{9+2.7}{57}\right) = 1.02\%$ (– ve)

(*b*) *Correction due to floor thickness*

From Fig. 11.11, it can be stated easily that the pressure at E_3 shall be less than at E_3', and since the pressure observed from curves is at E_3'; this correction shall be – ve and its amount

$$= \frac{38\% - 32\%}{152.0 - 141.7} \times 1.3 = \frac{16}{10.3} \times 1.3 = 0.76\% \text{ (– ve)}$$

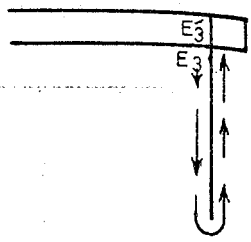


Fig. 11.11

(*c*) *Correction due to slope at* E_3 is nil, as the point E_3 is neither situated at the start nor at the end of any slope.

Hence, **corrected** ϕ_{E_3}

$$= 38\% - 1.02 - 0.76\% = \mathbf{36.22\%}$$ **Ans.**

The corrected pressures at various key points are tabulated below in Table 11.6.

Table 11.6

Upstream Pile No. 1	*Intermediate Pile No. 2*	*Downstream Pile No. 3*
$\phi_{E_1} = 100\%$	$\phi_{E_2} = 66.95\%$	$\phi_{E_3} = 36.22\%$
$\phi_{D_1} = 80.0\%$	$\phi_{D_2} = 63.0\%$	$\phi_{D_3} = 32.0\%$
$\phi_{C_1} = 74.38\%$	$\phi_{C_2} = 59.72\%$	$\phi_{C_3} = 0\%$

Exit Gradient

Let the water be headed up to pond level, *i.e.* on *RL* 158.0 m on the upstream side with no flow downstream.

The maximum seepage head $= H = 158.0 - 152.0 = 6.0$ m

The depth of d/s cut-off $= d = 152.0 - 141.7 = 10.3$ m

Total floor length $= b = 57.0$ m.

$$\alpha = \frac{b}{d} = \frac{57.0}{10.3} = 5.53$$

For a value of $\alpha = 5.53$, $\frac{1}{\pi\sqrt{\lambda}}$ from curves of Plate 11.2 is equal to 0.18.

Hence, $G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}} = \frac{6.0}{10.3} \times 0.18 = \frac{1}{9.53} = 0.105$

Hence, the exit gradient shall be equal to 0.105, *i.e.* **1 in 9.53**, which is very much safe.

Plotting the Hydraulic Gradient Line

The percentage pressures, computed and tabulated in Table 11.6, can be used to work out the elevation of H.G. line above the datum, as given in Table 11.7.

Table 11.7

Flow condition	Upstream water level in metres	Downstream water level in metres	Head in metres	Height/Elevation of Sub-soil H.G. Line above Datum								
				Upstream Pile Line			Intermediate Pile Line			Downstream Pile Line		
				ϕ_{E_1} 100%	ϕ_{D_1} 80%	ϕ_{C_1} 74.38%	ϕ_{E_2} 66.95%	ϕ_{D_2} 63.0%	ϕ_{C_2} 59.72%	ϕ_{E_3} 36.22%	ϕ_{D_3} 32.0%	ϕ_{C_3} 0%
Pond level u/s with no flow d/s	158.00	152.00	6.0	6.0	4.8	4.46	4.02	3.78	3.58	2.68	1.92	0.0
				158.0	156.8	156.46	156.02	155.78	155.58	154.17	153.92	152.0

The subsoil H.G. Line is then plotted in Fig. 11.12.

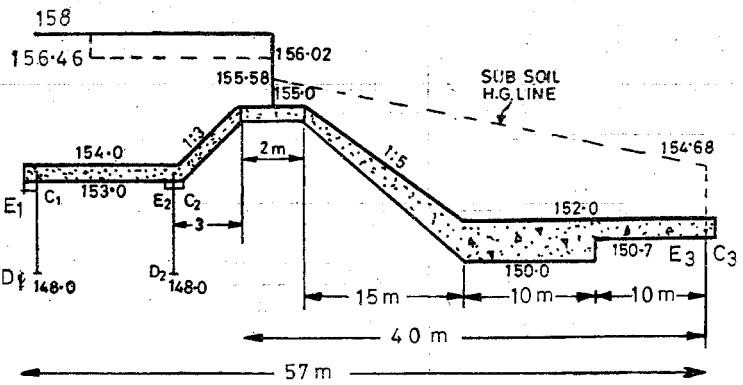


Fig. 11.12

Example 11.2. *In the previous example 11.1, the uncorrected percent residual pressures at C_1, D_1, E_2, D_2, C_2, E_3 and D_3, were all computed with the use of Khosla's charts. It is now desired to compare these pressures analytically by using the respective formulas, if the charts are not available.*

Solution. (1) **For Upstream Pile Line No. 1.**

$$b = 57 \text{ m}$$

$$d = 154.00 - 148.00 = 6.0 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57}{6} = 9.5.$$

Now, ϕ_E for such a case [Refer Fig. 11.5 (*a*)] is given by

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

where $\lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2}$

$$\therefore \quad \lambda = \frac{1+\sqrt{9.5^2}}{2} = 5.28$$

$$\therefore \quad \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{5.28-2}{5.28}\right) = \frac{1}{\pi} \times 51.65° \times \frac{\pi}{180°} = 0.287 \text{ ; } i.e. \text{ } 28.7\%.$$

$$\therefore \quad \phi_{C_1} = 100 - \phi_E = 100 - 28.7 = 71.3\% \textbf{ Ans.}$$

(as against 71% read out earlier)

Similarly, from Fig. 11.5 (*a*)

$$\phi_{E_1} = 100 - \phi_D$$

where $\phi_D = \dfrac{1}{\pi}\cos^{-1}\left(\dfrac{\lambda - 1}{\lambda}\right)$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{5.28-1}{5.28}\right) = 0.199 \text{ ; say } 19.9\%$$

$$\therefore \quad \phi_{D_1} = 100 - 19.9 = 80.1\% \textbf{ Ans.}$$

(as against 80% readout earlier)

(2) **Intermediate Pile Line No. 2.** As given in Fig. 11.5 (*d*),

$$\phi_{E_2} = \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$$

$$\phi_{C_2} = \phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 + 1}{\lambda}\right)$$

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

where $\lambda = \dfrac{\sqrt{1+\alpha_1^2} + \sqrt{1+\alpha_2^2}}{2}$

$$\lambda_1 = \frac{\sqrt{1+\alpha_1^2} - \sqrt{1+\alpha_2^2}}{2}$$

$$\alpha_1 = \frac{b_1}{d} \text{ ; } \alpha_2 = \frac{b_2}{d}.$$

From Fig. 11.8 of the given question,

d = Depth of intermediate pile
= 154.0 – 148.0 = 6 m

α_1 = Floor length *U/S* of intermediate pile
= 16.4 m.

α_2 = Floor length *D/S* of intermediate pile
= 40.6 m.

$$\therefore \quad \alpha_1 = \frac{16.4}{6} = 2.73$$

$$\alpha_2 = \frac{40.6}{6} = 6.77$$

$$\therefore \quad \lambda = \frac{\sqrt{1+2.73^2}+\sqrt{1+6.77^2}}{2} = \frac{2.907+6.843}{2} = 4.875$$

$$\lambda_1 = \frac{2.907-6.843}{2} = -1.968$$

Using, $\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1 - 1}{\lambda}\right)$

$$\phi_{E_2} = \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968-1}{4.875}\right) = 0.708 \text{ ; } i.e. \text{ } 70.8\%. \textbf{ Ans.}$$

(as against 70% read out earlier).

Now, using

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968}{4.875}\right) = 0.632 = 63.2\% \textbf{ Ans.}$$

(as against 63% read out earlier)

Now, using

$$\phi_{C_2} = \phi_C = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda_1+1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{-1.968+1}{4.875}\right) = 0.564 = 56.4\%.$$

(as against 56% read out earlier)

(3) **For Downstream Pile line No. 3.** W.R. to Fig. 11.5 (*b*)

$$b = 57.0 \text{ m}$$

$$d = 152.0 - 141.7 = 10.3 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{57}{10.3} = 5.53$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+5.53^2}}{2} = 3.31$$

$$\therefore \quad \phi_{E_2} = \phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{3.31-2}{3.31}\right) = \frac{1}{\pi}66.65° \times \frac{\pi}{180°} = 0.37 \text{ ; } i.e. \text{ } 37\%. \textbf{ Ans.}$$

(as against 38% read out earlier)

$$\phi_{D_2} = \phi_D = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-1}{\lambda}\right)$$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{3.31-1}{3.31}\right) = 0.254 \text{ ; } i.e., \text{ } 25.4\%. \textbf{ Ans.}$$

(as against 26% read out earlier)

Example 11.3. *An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the downstream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir*

floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory. (*U.P.S.C., Civil Services, 1990*)

Solution. In the given question, since Khosla's curves are not supplied, *it becomes necessary to remember and use* the formulas on which those Khosla's curves are based. These formulas are already mentioned in Fig. 11.5.

For the given question, pressures are required at inner junctions of both piles. Refer Fig. 11.5 (*a*) and (*b*). Pressures are thus required at C_1 for *U/S* pile, and at E at *D/S* pile line.

(*a*) **U/S Pile Line** ($\phi_{C_1} = ?$)

$$d = 4 \text{ m}$$

$$b = 16 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{16}{4} = 4$$

$$\phi_{C_1} = 100 - \phi_E$$

$$\phi_E = \frac{1}{\pi} \cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

where $$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} = \frac{1 + \sqrt{1 + 4^2}}{2} = 2.5616$$

$$\therefore \quad \phi_E = \frac{1}{\pi} \cos^{-1}\left(\frac{2.5616 - 2}{2.5616}\right) = \frac{1}{\pi} \times (77.34°)$$

$$= \frac{1}{\pi} \times (77.34°) \times \frac{\pi}{180°} = 0.4296 \text{ ; say } 0.43 \textit{ i.e. } 43\%.$$

$$\therefore \quad \phi_{C_1} = 100 - 43 = 57\%.$$

Correction due to D/S pile

$$C = +19 \cdot \left(\frac{D}{b'}\right)\left(\frac{d + D}{b}\right)$$

where D = Depth of D/S influencing pile = 5 m

d = Depth of U/S pile being influenced = 4 m

$b = b' = 16$ m

$$C = +19 \cdot \left(\frac{5}{16}\right)\left(\frac{4 + 5}{16}\right) = +6\%.$$

$$\therefore \quad \phi_{C1 \text{ (corrected)}} = 57\% + 6\% = 63\%$$

$$\therefore \quad P_{C_1} = 63\% \times 2.5 \text{ m} = \mathbf{1.575 \text{ m}}. \textbf{ Ans.}$$

(*b*) **For D/S Pile Line** ($\phi_E = ?$)

$$d = 5 \text{ m}$$

$$b = 16 \text{ m}$$

$$\alpha = \frac{b}{d} = \frac{16}{5} = 3.2$$

$$\lambda = \frac{1 + \sqrt{1 + 3.2}}{2} = 2.176$$

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{2.176-2}{2.176}\right)$$

$$= \frac{1}{\pi} \times 85.35° \times \frac{\pi}{180°} = 0.474 = 47.4\%.$$

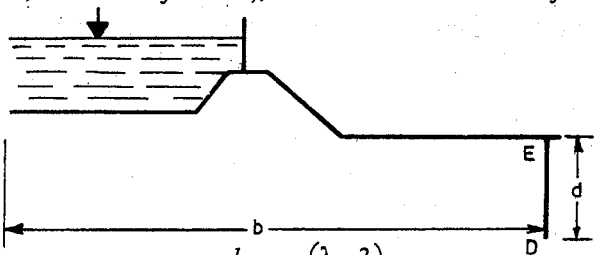
Correction due to U/S pile.

$$C = (-)\ 19 \cdot \sqrt{\frac{4}{16}} \times \left(\frac{5+4}{16}\right) = -5.3\%$$

$$\phi_{E\,(corrected)} = 47.4 - 5.3 = 42.1\%$$

$$P_E = 42.1\% \times 2.5\ \text{m} = 1.05\ \text{m}. \quad \textbf{Ans.}$$

Example 11.4. *The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off walls at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that of the downstream wall is 2.0 m. Using Khosla's theory (see Fig. 11.13 for definition, sketch and formula), determine the thickness of the floor at its*



$$\phi_E = \frac{1}{\pi} cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d}$$

Fig. 11.13. Definition sketch for Khosla's theory for uplift pressures.

mid length and also at its junction with the upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm anywhere. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ? *(U.P.S.C. Civil Services, 1982)*

Solution. (*i*) For U/S cut off wall.

W.r. to Fig. 11.5 (*a*) and (*b*), we have

$b = 13$ m

d = depth of pile line from top of floor level = 1.5 m.

$$\alpha = \frac{b}{d} = \frac{13}{1.5} = 8.67$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} = \frac{1+\sqrt{1+8.67^2}}{2} = 4.86$$

Now, $\phi_{C_1} = 100 - \phi_E$

where $\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$

$$= \frac{1}{\pi}\cos^{-1}\left(\frac{4.86-2}{4.86}\right) = \frac{1}{\pi} \times 53.9° \times \frac{\pi}{180°} = 0.30 \text{ ; } i.e., 30\%$$

$$\therefore \quad \boxed{\phi_{C_1} = 100 - 30 = 70\%.}$$

(*i*) *Correction for D/S pile line.*

$$\text{Correction} = 19 \cdot \sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

where D = Depth of influencing D/S pile = 2 m
d = Depth of U/S pile getting influenced = 1.5 m
b' = Distance between two piles = 13

$$= (+)\ 19 \cdot \sqrt{\frac{2}{13}}\left(\frac{1.5+2}{13}\right) = 2.0$$

(Since point C_1 is in the rear in the direction of flow, the correction is positive)

(*ii*) *Correction for floor thickness.* Strictly speaking, this **positive** correction needs to be worked out by assuming certain floor thickness of say 1 m at U/S end ; but for that, Khosla's formula for ϕ_{D_1} or $\phi_{D_1} = 100 - \phi_D$; where $\phi_D = \frac{1}{\pi}\cos^{-1}\frac{\lambda-1}{\lambda}$ is required, which is not given in the question, although the other one for ϕ_E is given, which shows that the examiner wants us to ignore this correction. If time permits, this correction can also be worked out.

Hence, **corrected** ϕ_{C_1}

$$= 70 + 2 = 72\% \qquad \text{(Ignoring + ve correction due to floor thickness)}$$

(*a*) P_{C_1} = Residual pressure causing uplift at start point (inner edge of U/S cutoff)

$$= \text{Corrected } \phi_{C_1} \times 1.5 = 72\% \times 1.5 = 1.08 \text{ m}$$

$\therefore$ **Depth of floor required at start point C_1**

$$= \frac{1.08}{G-1} = \frac{1.08}{1.24} = 0.87 \text{ m ; say 1 m}$$

Use 1.0 m depth (to be conservative for not accounting + ve correction for floor thickness). **Ans.**

(2) **For D/S cutoff wall**

$$b = 13 \text{ m}$$
$$d = 2 \text{ m}$$
$$\alpha = \frac{13}{2} = 6.5$$
$$\lambda = \frac{1+\sqrt{1+6.5^2}}{2} = 3.79$$
$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda-2}{\lambda}\right)$$

$$= \frac{1}{\pi} \cos^{-1}\left(\frac{3.79-2}{3.79}\right) = 0.3434 \text{ ; say } 34.34\%.$$

Correction for the effect of U/S pile line. It is – ve, as the point is forward in the direction of flow

$$= (-)\ 19 \cdot \sqrt{\frac{D}{b'}} \cdot \left(\frac{d+D}{b}\right) = (-)\ 19 \cdot \sqrt{\frac{1.5}{13}} \left(\frac{2+1.5}{13}\right) = (-)\ 1.74\%.$$

$\therefore$ **Corrected** $\phi_E = 34.34 - 1.74 = 32.6\%$.

(*b*) P_E = Residual pressure causing uplift at end point (inner edge of D/S cutoff)
= Corrected $\phi_E \times 1.5 = 32.6\% \times 1.5 = 0.489$ m

Thickness of floor required at end point (E)

$$= \frac{0.489}{1.24} = 0.394, \text{ say } 0.4 \text{ m. } \textbf{Ans.}$$; which eventually is more than the min. specified of 0.3 m

(*c*) **Thickness of floor at mid length** can be the average of the two thicknesses, because uplift is varying from start to end, from 72% to 32.6% ; and its value at mid length is just average of the two. Hence, use floor thickness at mid point as

$$= \frac{1.0+0.4}{2} = \textbf{0.7 m. Ans.}$$

Hence, floor thickness at start = 1.0 m. **Ans.**
floor thickness at end = 0.4 m. **Ans.**
floor thickness at mid length = 0.7 m. **Ans.**

(3) **Exit Gradient.** Exit gradient is given by Eqn. 11.4 as

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

where H = Total head = 1.5 m (given)

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d}$$

d = Depth of D/S cutoff = 2 m

$$\alpha = \frac{b}{d} = \frac{13}{2} = 6.5$$

$\therefore$ $$\lambda = \frac{1+\sqrt{1+6.5^2}}{2} = 3.79,$$

$\therefore$ $$G_E = \frac{1.5}{2} \frac{1}{\pi \cdot \sqrt{3.79}} = \textbf{0.123} < \textbf{0.18} \text{ (Permissible)}$$

Hence, the floor is safe against piping. **Ans.**

11.5. Design of a Vertical Drop Weir on Bligh's Theory

Many of the vertical drop weirs, such as shown in Fig. 9.6, have been designed on Bligh's theory ; and even though this theory has now been replaced by modern Khosla's theory, yet it

is still used at certain places and especially for minor works, owing to its simplicity. The design of a vertical drop weir, on the basis of this theory is explained below :

Design of Pucca-floor and Aprons. As discussed earlier the total length of the pucca floor of the weir (including twice the length of cut-off, if provided) is designed in accordance with the equation $L = CH_L$, and the thickness of the floor is designed by using the equation $t = 1.33\left(\frac{h}{G-1}\right)$. Bligh has further given certain empirical formulas for determining the length of the downstream pucca floor (L_2). The cut-offs may be provided as per the provision of Khosla's theory (explained in article 11.4). The balance floor length [*i.e.* (the total length) – (the downstream length + twice the cut-off length)]

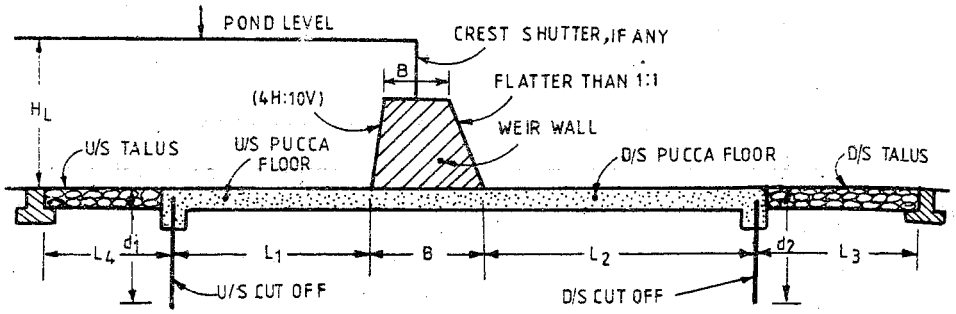


Fig. 11.14. Dimensions of Vertical Drop Weir based on Bligh's Theory.

is then provided under the crest and on the upstream side as shown in Fig. 11.14. The lengths of the upstream and downstream loose stone talus* (or aprons) which are provided in order to prevent the scour from reaching the pucca floor are also worked out by the empirical formulas put forward by Bligh.

The various empirical formulas put forward by Bligh (w.r.t. Fig. 11.14) are given below :

(*a*) $L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{13}}$ for weirs having crest shutters. ..(11.5)

(*b*) $L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{10}}$ for weirs having no crest shutters . ..(11.6)

where H_L = the total head loss.

L_2 = the length of d/s pucca floor.

(*c*) $L_2 + L_3 = 18\, C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$ for weirs having crest shutters ...(11.7)

(*d*) $L_2 + L_3 = 18\, C \cdot \sqrt{\frac{H_L}{10} \cdot \frac{q}{75}}$ for weirs having no crest shutters. ...(11.8)

---

* In continuation to the upstream pucca floor, a length L_4 (as shown in Fig. 11.14) of loose stone talus is provided to keep the erosion and scour away from the upstream pucca floor. Similarly, a length L_3 of loose sstone talus is provided on the downstream of pucca floor, so as to dissipate the residual energy and to keep the scour away from the pucca floor. As soon as the scour occurs, the talus falls into the scour, and thus preventing the scour to travel and reach upto the pucca floor. The rational design of the talus has been discussed a little later, under "Design of Protection Works", in article 11.6.1.

where q = the discharge intensity in cumecs/metre.
L_3 = the length of d/s loose stone talus.

The length of upstream talus (L_4) may be kept equal to half the length of d/s talus. Thus

$$L_4 = \frac{L_3}{2} \qquad ...(11.9)$$

The above formulas are applicable for designing the proper weir portion ; whereas for designing the 'undersluice' portion of the weir, the following modified formulas are used.

(*i*) $L_2 = 3.87\, C \cdot \sqrt{\frac{H_L}{13}}$ for 'undersluices' having crest shutters . ..(11.10)

(*ii*) $L_2 = 3.87\, C \cdot \sqrt{\frac{H_L}{10}}$ for 'undersluices' having no crest shutters . ..(11.11)

(*iii*) $L_2 + L_3 = 27\, C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$ for 'undersluices' having crest shutters ...(11.12)

(*iv*) $L_2 + L_3 = 27\, C \cdot \sqrt{\frac{H_L}{10} \cdot \frac{q}{75}}$ for undersluices having no crest shutters ...(11.13)

Design of Weir Wall. Bligh has further given certain empirical formulas for the design of weir wall. According to him, the top width of weir wall (B') is given as :

$$B' = \frac{H}{\sqrt{G-1}} \qquad ...(11.14)$$

where B' = Top width of weir wall and is generally 1.5 to 1.8 m.
H = Head of water over the weir wall at the time of max. flood.
G = Specific gravity of floor material.

Further, the crest width should also be greater by 0.6 m than the height of the crest shutters ; if any.

The bottom width (B) of the weir wall may be obtained by providing suitable side slopes. The u/s batter may by kept as $4H : 10V$ and the d/s batter should not be flatter than 1 : 1. The bottom width (B) of the weir wall should not be less than

$$B = \frac{H + \text{Height of weir}}{\sqrt{G-1}} \qquad ...(11.15)$$

The crest level of the weir wall and the height of solid masonry weir is determined from the considerations of afflux. The afflux produced should not exceed the allowable value, generally kept less than 1.5 m or so. If the crest level, works out to be practically equal to the pond level, then a solid masonry weir can be provided ; and if it is much less than the pond level, then the balance may be provided by crest shutters.

Example 11.5. *A weir with a vertical drop has the following particulars :*

Nature of bed : Coarse sand with the value of Bligh's C = 12
Flood discharge = 300 cumecs

Length of weir = 40 m
Height of weir above low water = 2 m
Height of falling shutter = 0.6 m
Top width of weir = 2.0 m
Bottom width of weir = 3.5 m

Design the length and thickness of aprons and draw the cross-section of the weir.

(Madras University, 1975)

Solution. Total max. head loss $= H_L = 2 + 0.6 = 2.6$ m

Total length of creep required including creep along cut-off

$$= L = C.H_L = 12 \times 2.6 = 31.2 \text{ m.}$$

The length of downstream floor is given by Eq. (11.5) as

$$L_2 = 2.21\, C \cdot \sqrt{\frac{H_L}{13}}$$

or $$L_2 = 2.21 \times 12 \cdot \sqrt{\frac{2.6}{13}} = 2.21 \times 12 \times 0.447 = 11.8 \text{ m; say } 12 \text{ m}$$

The bottom width of weir $= B = 3.5$ m.

Provision of cut-offs

Let us first calculate as to what will be the head over the weir when high flood discharge is passing.

Use $q = 1.7\, H^{3/2}$

where $q = \frac{Q}{L} = \frac{300}{40} = 7.5$

$\therefore$ $7.5 = 1.7\, H^{3/2}$

$\therefore$ $H^{3/2} = \frac{7.5}{1.7} = 4.41$

or $H = (4.41)^{2/3} = 2.68$ m

Head over the weir crest = **2.68 m.**

$\therefore$ U/s HFL (assuming bed level as 100.0 m) and crest level as 102 m)

$$= 102.0 + 2.68 = \mathbf{104.68\ m}$$

Now R = Lacey's regime scoured depth $= 1.35\left(\frac{q^2}{f}\right)^{1/3}$

assuming $f = 1$

$$R = 1.35\left(\frac{7.5^2}{1}\right)^{1/3} = 1.35 \times 3.84 = \mathbf{5.18\ m}$$

Depth of u/s sheet pile from below u/s HFL

$$= 1.5\, R = 1.5 \times 5.18 = 7.8 \text{ m}$$

$\therefore$ Level of bottom of U/s sheet pile

$$= 104.68 - 7.8 = 96.88 \text{ m}$$

Provide a depth of $100 - 96.88 = 3.02$, say 3 m for u/s cut-off.

Since afflux is not known, the d/s HFL is not known; and the exact calculation for the depth of d/s sheet pile cannot be done. Use a similar d/s cut off of 3 m depth below the weir floor.

Now, total creep length provided except u/s floor

$$= 2 \times 3 \text{ m} + 3.5 \text{ m} + 12 \text{ m} + 2 \times 3 \text{ m}$$
$$= 27.5 \text{ m}$$

The balance length, *i.e.* $31.2 - 27.5 = 3.7$ m; say 5 m; is therefore, provided as u/s floor as shown in Fig. 11.15. Hence, total creep length provided $= 27.5 + 5 = 32.2$ m.

Now using Eq. (11.7), we have

$$L_2 + L_3 = 18\, C \cdot \sqrt{\frac{H_L}{13} \cdot \frac{q}{75}}$$

where L_3 is the length of downstream loose talus

$$q = \text{Discharge intensity} = \frac{Q}{L} = \frac{300}{40} = 7.5$$

$$L_2 + L_3 = 18 \times 12 \sqrt{\frac{2.6}{13} \cdot \frac{7.5}{75}}$$

$$= 18 \times 12 \times 0.141 = 31.6 \text{ m}$$

But $L_2 = 11.8$ m

$\therefore$ $L_3 = 31.6 - 11.8 = \mathbf{19.8\ m}$; say 20 m.

Hence provide (say 1 m thick) d/s loose talus of 20 m in length.

(**Note** : This length of 20 m can be partly provided as blocks over inverted filter and partly as launching apron. The length of u/s talus L_4 may be kept as equal to $\frac{1}{2}$ the length of d/s talus, *i.e.* 10 m.

D/s Floor Thickness

The H.G. line is now plotted as shown in Fig. 11.15. The max. ordinate of the H.G. line above the bottom of the floor for the downstream portion at the junction of weir wall

$$= h = \frac{2.6}{32.3} \times 18 = 1.45 \text{ m}$$

The thickness of D/s floor at this point is then obtained by Eq.

$$t = 1.33\left(\frac{h}{G-1}\right) = 1.33 \cdot \left(\frac{1.45}{2.4-1}\right)$$

$$= \frac{1.33 \times 1.45}{1.4} = 1.35; \text{ say } \mathbf{1.4\ m}.$$

Hence, provide 1.4 m thickness for D/s floor from just near its junction with weir wall. The thickness can be curtailed as below.

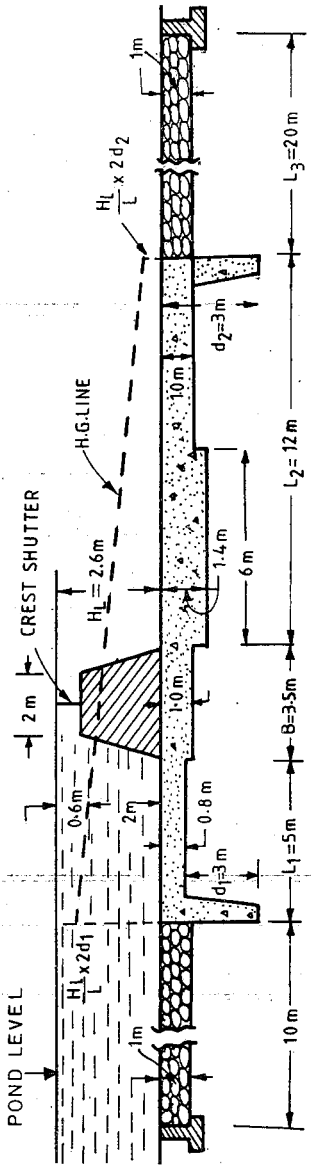


Fig. 11.15

The thickness required at half way of D/s floor length

$$= \frac{1.33\,h}{1.4}$$

where $h = \frac{2.6}{32.3} \times 12 = 0.97$ m

$$= \frac{1.33 \times 0.97}{1.4} = 0.92 \text{ m; say 1 m.}$$

Further, provide a nominal thickness of 0.8 m below the u/s floor and 1 m below the weir wall. The complete details are shown in Fig. 11.15.

11.6. Design of Modern Weirs and Barrages Founded on Permeable Foundations on the Basis of Khosla's Theory

The complete design of a modern glacis-weir or a barrage can be divided into two main aspects, *i.e.*

(1) Hydraulic Design ; and

(2) Structural Design.

11.6.1. Hydraulic Design for the Weir. The 'hydraulic design' involves determining the section of the weir and the details of its u/s cutoff, crest, glacis, floor, d/s cutoff, protection works u/s and d/s, etc. The hydraulic design of weirs on permeable foundations may be classified into :

(*i*) *Design for Sub-surface Flow;* and

(*ii*) *Design for Surface Flow.*

The effects of sub-surface flow or seepage flow on the stability of a hydraulic structure founded on permeable foundations have been thoroughly described earlier and the same hold good for weirs or barrages. *Khosla's method of independent variables is invariably used for determining the uplift pressures exerted by the seeping water on the floor of the weir. The safety of the structure against piping has to be checked by keeping the exit gradient within safe limits.*

The maximum uplift pressure shall occur when the pond is full and there is no water flowing down the weir. But when flood water passes over the weir, entirely new conditions are superimposed. The formation of hydraulic jump causes uplift or unbalanced head in the jump trough, which may be larger than that under steady seepage, as explained below.

Uplift pressure in the jump trough. The maximum difference of head and hence the maximum uplift pressures are normally imposed on the structure when water is ponded upto the highest level on the upstream side without any discharge passing down the weir. The hydraulic gradient line under such a situation shall be as shown in Fig. 11.16 (*a*).

The pressure distribution along the length of the weir section is also shown in Fig. 11.16 (*a*).

When a certain discharge is passing over the

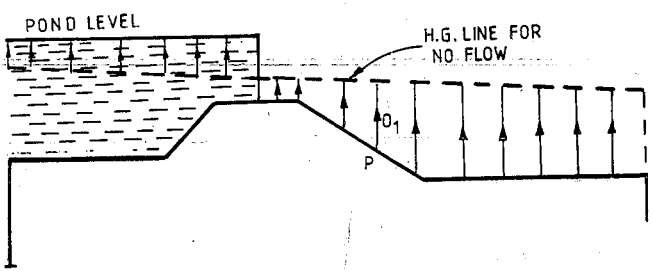


Fig. 11.16 (*a*). Uplift pressures in jump trough for no flow.

weir and hydraulic jump is forming, the seepage head is the difference of the water level of upstream and downstream, which is generally much smaller than the seepage head in case there is no flow. The hydraulic gradient line for this case has also been plotted along with water surface profile, as shown in Fig. 11.16 (*b*).

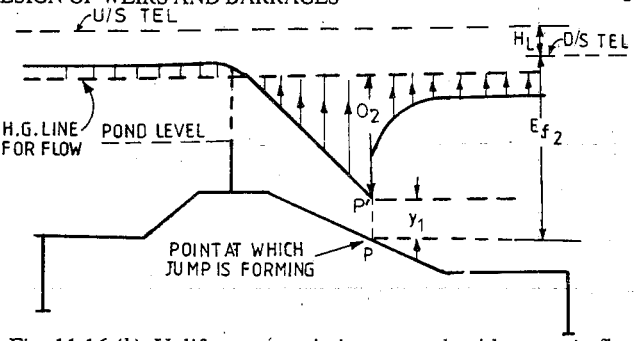


Fig. 11.16 (*b*). Uplift pressures in jump trough with a certain flow.

The ordinate of the uplift pressure is to be measured from the H.G. line to the water surface, as the rest of the uplift is exactly counterbalanced by the weight of the water standing on the floor. The pressure distribution is also shown in Fig. 11.15 (*b*). It is quite evident from these figures that the uplift pressure on the upstream side is more than counterbalanced by the water standing on the floor. Thus, it becomes very clear that there is absolutely no uplift acting on u/s floor, and hence, only a nominal floor thickness is to be provided on the upstream side, and no design is required for u/s floor thickness.

Further, if Fig. 11.16 (*a*) and (*b*) are imposed upon each other, it is found that the uplift ordinate O_2 in the second case is much larger than the uplift ordinate O_1 in the first case at the corresponding point P; where the jump is forming and also at some other points in its vicinity. Since the point of jump formation P is likely to shift with the variation in discharge passing over the weir, the entire glacis may have to be designed for the second condition; while most of the remaining floor may have to be designed for the first condition. In other words, the requirement of floor thickness should be obtained by taking the larger of the two ordinates, and dividing it by the submerged density of the floor material, *i.e.* $(G-1) = (2.24-1) = 1.24$ for concrete.

The water surface profile after the jump and before the jump can be plotted as explained in Chapter 10, and then, H.G. line can be plotted and the net ordinate determined easily. The floor thickness shall have to be designed to withstand the larger pressure.

The uplift pressure due to dynamic action (*i.e.* hydraulic jump) are further reduced due to the following reasons :

(*i*) The backward rolling flow of water in the trough

(*ii*) The uplift pressure due to jump is maximum at the point of the jump but reduces rapidly on either side. As the floor has beam action, it may be designed for average uplift ordinate rather than for maximum uplift ordinate.

(*iii*) The vertical component of the momentum remains unaffected in the jump, which exerts a downward pressure in the vertical direction.

Due to these reasons, the uplift pressure due to dynamic action is reduced to $\frac{2}{3}$*rd of the theoretical value, and the design is done for maximum of either the* $\frac{2}{3}$*rd of the largest ordinate due to dynamic action or the largest ordinate due to steady seepage.*

Uplift pressures at the point of jump formation may also approximately be taken as $= [50\% (y_2 - y_1) + \phi \cdot H_L]$, *where* ϕ *is the percentage of pressure at the jump location. (Generally used in the design of falls, etc.)*

The other aspects of surface flow are scour considerations. The sheet piles at the ends must go below the deepest anticipated scour level.

For a discharge intensity q, the normal depth of scour (R) is given by Lacey's equation (4.20) *i.e.*,

$$R = 1.35\left[\frac{q^2}{f}\right]^{1/3}$$

The value of q will be different for the weir and for the undersluice section, and should, therefore, be taken separately for each. For the design of sheet piles, it is just enough to take them down to the level obtained by measuring the normal depth of scour R, below the H.F.L. Though sometimes, even 1.5 R on the upstream side and 2.0 R on the d/s side is taken in conservative designs. A value of 1.25 R on u/s side and 1.5 R on d/s is widely accepted as a *via media.*

Length of Pucca Concrete Floor. The total length of the pucca floor is mainly governed by the exit gradient considerations. For a safe exit gradient and a depth of downstream cut-off suitable from scour considerations, the length of the horizontal floor 'b' can be worked out as $b = \alpha \cdot d$ (α is known when G_E is fixed).

The main turbulence of the hydraulic jump is generally confined to a length equal to five times the jump height. Hence, a pucca floor equal to or more than 5 $(y_2 - y_1)$ in length is provided after the lowest point of jump formation, *i.e.* the end point of glacis. *The glacis should be sloped down at a slope of 3 : 1 to 5 : 1 for maximum dissipation of energy coupled with economy and stability of the jump.* The top width of the crest is generally kept as 2.0 to 3.0 metres from practical considerations, and the upstream slope to the crest is kept as 1 : 1 to 3 : 1. The length of the upstream floor may be adjusted so as to provide the necessary total floor length b, calculated above, as $b = \alpha.d$.

By providing a deeper d/s cut-off, it is possible to reduce the floor length 'b' and *vice-versa*, and hence b and d can be mutually adjusted to provide the most economical and suitable combination and to keep a safe exit gradient.

Design of Protection Works. Protection works are required on the upstream as well as on the downstream in order to obviate the possibility of scour hole travelling close to the pucca floor of the weir and to relieve any residual uplift pressure through the filter. The arrangement consists of (*i*) Inverted filter, and (*ii*) Launching apron, as explained below.

(*A*) *Downstream Loose Protections*

(*i*) **Inverted filter.** Just after the end of the concrete floor, an '*Inverted filter*', 1.5 to 2 D long is generally provided, where D is the depth of scour below the original river bed. The total scour below HFL is taken as xR, where R is Lacey's normal scour depth and values of x for different classes of scour are tabulated below in Table 11.8. *Value of x is generally taken as 1.5 for design of d/s protection works and 1.25 for design of u/s protection works.*

Table 11.8

Class of scour	*Reach*	*Mean value of x*	*D = xR – Water depth above bed*
A	Straight	1.25	$1.25R - y$
B	Moderate Bend	1.50	$1.50R - y$
C	Severe Bend	1.75	$1.75R - y$
D	Right-angled Bend	2.00	$2R - y$

The depth of inverted filter is kept equal to the depth of d/s *launching apron.* It generally consists of 1.0 to 1.2 m deep concrete blocks with open joints laid over 0.6 m thick graded filter material. The openings between the blocks are filled with *clean bajri.*

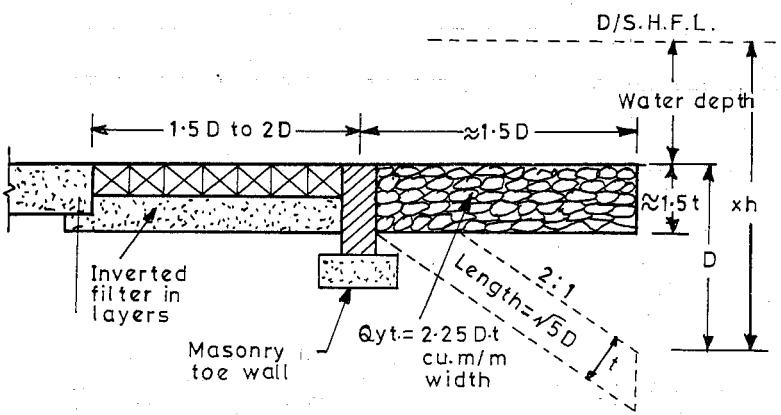


Fig. 11.17. Details of d/s loose projections.

An 'inverted filter' invariably reduces the possibility of piping, as it allows free flow of seepage water through itself without allowing the foundation soils to be lifted upward. *The filter, therefore, consists of layers of materials of increasing permeability from bottom to top.* The gradation should be such that while it allows free flow of seepage water, the foundation material does not penetrate to clog the filter. The design criteria to satisfy these conditions are discussed in chapter 20 on "Earth Dams". To prevent filter from dislocation under surface flow, concrete or masonry blocks are laid over the filter material.

(*ii*) **Launching apron.** After the inverted filter, the loose apron called 'launching apron' is provided for a length, generally equal to 1.5 *D*, where *D* has the same meaning as given above in inverted filter calculations.

The apron generally launches to a slope of 2 : 1, and if *t* is the thickness of the apron in the launched position, length being $\sqrt{5}\,D$; the volume of stone per metre width will then be

$$= \sqrt{5} \cdot D \cdot t = 2.24\,D.t.;$$ or **say 2.25 D.t.**

Hence, Volume of stone per metre is given by 2.25 *D.t.*cu. m/metre.

Since the volume of stone should be the same in launched and unlaunched apron, and if the unlaunched apron is laid in a length equal to 1.5 *D*, the thickness of the unlaunched apron is given as :

$$= \frac{2.25\, D.t}{1.5\, D} = 1.5\, t$$

If t is taken as 1 m ; then an apron of 1.5 m thickness and 1.5 D metres long has to be laid as d/s launching apron. Different values of t have been recommended by different investigators.

Blench has recommended t equal to 1.5 to 2 times the size of the stone (d_s) required by equation (11.15) as :

$$V_{av} = 4.915\, d_s^{1/2} \qquad ...(11.15)$$

where V_{av} = average velocity of flow in m/sec.

d_s = mean diameter of stone in metres.

Spring has recommended t as about 0.9 m. Gales recommended t varying from 1.35 to 1.9 m, for discharge varying from 7,000 to 70,000 cumecs. Blench's recommendations are quite adoptive.

(B) Upstream Loose Protections

Just upstream of the concrete floor of the weir, block protection is provided. It generally consists of concrete blocks laid over packed stone, for a length equal to D. ($D = xR - y$, where $x = 1.0$ to 1.5, generally taken as 1.25).

Upstream of the blocks, a launching apron is provided in the same way as described for the downstream portion, except that the proper value of x should be chosen.

Toe walls are always generally constructed in between the 'filter' and the 'apron' as shown in Fig. 11.17.

11.6.2. Structural Design for the Weir Floor. The concrete floor is usually designed for the uplift pressures as a pure gravity section at each point. Sometimes, when H_L is enormous, the uplift pressures are too high, and as such, the thickness of the required concrete gravity section becomes enormous and, therefore, becomes very costly. In such cases, the floor may be designed as a reinforced concrete raft structure held down by the weight of the raft and piers. A raft, in such cases, may be cheaper and more desirable as the thin section of raft reduces deep excavations and dewatering problems. Recently, Farraka Barrage, Narora Barrage, and Durgapur Barrage have been designed and constructed as reinforced raft structures. However, the treatment of this type of design is a simple structural problem. The total downward weight of raft and piers is counterbalanced by a uniform uplift, and design is done by working out bending moments, etc.

11.6.3. Effects Produced by Weir on River Regime and Retrogression of Downstream Levels. Before we start with the actual design of a weir, let us first review the effects that are produced by the weir construction, on the regime of the river. The first effect produced by the construction of a weir across a river, is that : *the downstream bed of the river goes on eroding, consequently causing progressive lowering of the downstream levels. This progressive lowering of the downstream levels is known as Retrogression of downstream levels or* **retrogression.**

The basic cause for retrogression is the variation in the silt carrying capacity of the channel. As soon as a weir is constructed, the water starts ponding on its upstream side, causing the water surface slope to flatten for some distance behind the weir. This reduces the silt carrying capacity of the river in this reach, and consequently the silt deposition starts, *i.e.* the river starts dropping sediment, and this leads to formation of shoals and

islands on the upstream side. This clearer water passes over the weir and picks up sediment from its downstream bed, so as to fulfil the increased demand of the silt carrying capacity of the channel in the downstream, *i.e.* the sediment deficit caused on the upstream is made up by eroding extra sediment from the downstream. This causes the progressive lowering of the downstream bed levels. The process continues for a number of years till the river starts to regain its original slope in the upstream portion by extending the afflux more and more upstream. The stage is gradually reached, when the upstream pond absorbs no more silt. As the off-taking channel takes comparatively silt free water, the sediment will go downstream, while the discharge going down will be below normal, and hence, the sediment taken shall be more than the 'carrying capacity of the river' ; consequently resulting in sediment deposition on the down stream river bed and long range recovery of d/s bed levels.

A provision must, therefore, be made for retrogression of d/s bed, while designing a weir, as it shall lower the d/s TEL and increase H_L. Hence, if retrogression is not taken into account, it may lead to undermining of floor. Observations on various weirs in Punjab have exhibited a retrogression of 1.2 to 2.2 metres. The retrogression of such a high magnitude has been observed at low water levels, but at high flood levels, the maximum retrogression is between 0.3 to 0.5 metres.

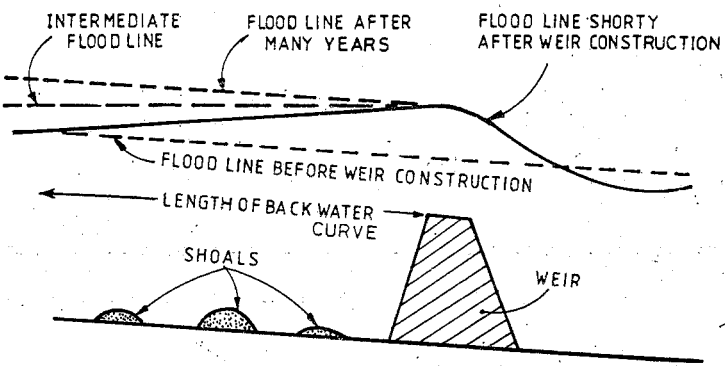


Fig. 11.18. Figure showing the effects of weir construction on flood levels.

Hence, a retrogression of 0.5 m at high flood stage, and a higher retrogression varying linearly upto 2.0 m at lower discharges, is generally considered in the design of weirs or barrages.

The recovery of downstream bed levels, sometimes continues even beyond the original bed levels. This may lead to reduced control on silt regulation. Hence, sufficient margin must be provided between the canal full supply level and the pond level, so as to allow raising of the crest of the canal Head Regulator, if found necessary, in future.

Since the afflux extends on the upstream of the weir with the passage of time, it increases the high flood levels on the upstream even beyond the original backwater curve. Hence, the marginal bunds will have to be extended upstream as soon as the above effects come into picture. Since it happens after many years, it is economical to construct marginal afflux bunds only for the backwater length in the beginning, and to extend them upstream afterwards.

11.6.4. Factors Governing the Design of a Weir or a Barrage The following data must be collected before a weir or a barrage can be designed :

(*i*) High flood levels for the river at the weir site.

(*ii*) High flood or maximum flood discharge for the river at the weir site.

(*iii*) River cross-section at the weir site.

(*iv*) The stage discharge curve for the river at the weir site.

All these informations can be obtained from Topographical maps of the area and by consulting the Hydrological and Meteorological Departments of the area.

Factors to be Decided. Besides Retrogression, certain other factors which have to be decided while designing a weir or a barrage are :

(*i*) Crest Levels.

(*ii*) Afflux.

(*iii*) Waterway and the discharge per metre.

(*iv*) Pond Level.

They are described below :

(*i*) **Crest Levels.** It has been stated earlier that the weir consists of two parts :

(*a*) The main weir section, called Weir Bay Section.

(*b*) Undersluice Section.

The undersluice section is kept at a lower level, so as to provide deeper silent river pocket near the canal head regulator. The undersluice crest is, therefore, kept slightly lower than the barrage bay crest so as to attract a deep current infront of the canal head regulator, so that dry weather flow may remain rear the regulator.

The undersluice crest is generally kept as near the bed level in the existing deepest channel, as is practically possible.

The crest level of other barrage bays is generally kept 1.0 to 1.5 metres higher than the crest level of the undersluices. It is also guided by the general bed level of the river in the barrage bay portion.

It can be easily seen that the afflux and discharge per metre are dependent upon the crest levels. If crest level is low, afflux shall be less, and since the depth of water over the crest will be more, it shall lead to higher discharge per metre. A low set barrage, with increased depth of water over the crest may, therefore, result in the increase in height of gates, thickness of floor, and cost of superstructure above floor level.

(*ii*) **Afflux.** It was defined earlier that the rise in the maximum flood level of the river upstream of the weir after construction is known as afflux. This afflux is confined only to a short reach (equal to the length of the back water curve, in the beginning, but extends gradually very far up, as explained earlier.

The amount of afflux will determine the top levels of guide banks and marginal bunds. The lengths and sections of marginal bunds are also dependent upon it. It will govern the dynamic action downstream of the work as well as the depth and location of the hydraulic jump. *By providing a higher afflux, the waterway and, therefore, the length of the weir can be reduced, but it will increase the cost of training works and the risk of failure by outflanking.* At the same time, the discharge intensity and the consequent scour shall go up, and hence, the *sections of loose protections upstream and downstream as well as the depths of pile lines at either ends shall have to be increased,* thereby making it costly. *It is, therefore, always desirable to limit the afflux to a safe value of*

1.0 to 1.2 metres, more commonly 1.0 metre. However, in steep reaches with rocky bed, a higher value of afflux may be permitted.

(iii) **Waterway and discharge per metre.** The waterway and afflux are correlated. If afflux is increased, waterway is reduced and *vice-versa.* Hence, a limit placed on maximum afflux shall limit the minimum waterway. It shall be seen that the cost of works as a whole is minimum for a certain waterway and afflux. Attempts should, therefore, be made to attain the most economical combination of these two factors. This can be made by trial and error, generally limiting the maximum value of afflux.

A likely figure for the waterway is obtained by Lacey's wetted perimeter formula, given by eq. (4.18) as

$$P = 4.75\sqrt{Q}$$

A waterway equal to 1.2 to 1.4 P is generally assumed in rivers in plains. Some engineers preferred to keep a shorter waterway inspite of costlier works, as it was thought that a shorter waterway reduces shoaling, but as a matter of fact, it is not so.

(iv) **Pond levels.** The pond level is the minimum water level required in the undersluice pocket upstream of the canal head regulator, so as to feed the canal with its full supply. *The pond level is generally obtained by adding 1.0 to 1.2 metres to the canal FSL.*

Water in the undersluice pocket has to be maintained at Pond Level, even during dry weather flow. This can be accomplished either by a raised crest or by shutters or by a combination of both. A permanently raised crest will lead to higher afflux during floods and is likely to result in loss of control on the river.

In modern design of a barrage, the entire ponding is done by gates which are opened during floods and the crest level of the undersluices is generally taken as the available river bed level in the deepest channel. *No raised crest is thus generally provided for the undersluices.* A raised crest is provided where possible, as it improves the coefficient of discharge.

The **discharge formulas** to be used in the design of a weir or a barrage are

(*a*) **For a broad crested weir :**

$$Q = 1.7\,(L - Kn.H)\,H^{3/2} \qquad \text{...(11.16)}$$

where Q = Discharge in cumecs

H = Total head in metres including velocity head

n = No. of end contractions (Twice the number of gated bays)

L = Clear waterway length in metres.

K = coefficient of end contraction varying from 0.1 for thick blunt peir noses to 0.04 for thin pointed noses : generally taken as 0.1 in ordinary calculations.

(*b*) **For a sharp crested weir :**

$$Q = 1.84 \cdot (L - KnH)\,H^{3/2}.$$

Q, L, n, K and H have the same meanings as given above.

If the head over the weir crest is more than 1.5 times the width of the weir, the weir behaves as a *sharp crested weir.*

Concentration factor. While calculating the cistern levels and the depths of cutoffs, the possibility of non-uniform flow is taken into account by providing a suitable concentration factor. This factor is chosen arbitrarily. Generally, a 20% concentration is taken at any particular section. Hence, the calculated maximum discharge intensity is increased by 20% in designs. No concentration of flow is taken while designing protection works.

Example on the design of a Barrage

Example 11.6. *A barrage is to be constructed on a river having a high flood discharge of about 8,100 cumecs, with the given data as follows :*

Average bed level of the river = *257.0 m*
High Flood Level (before construction of barrage) = *262.2 m*
Permissible afflux = *1.0 m*
Pond Level = *260.6 m.*

Stage Discharge curve for the river at the barrage site is given in Fig. 11.19. Prepare a complete hydraulic design for the undersluice section as well as for other barrage bay section, on the basis of Hydraulic jump theory and Khosla's theory. A safe exit gradient of 1/6 may be assumed. 0.5 metres retrogression and 20% discharge concentration may be assumed where non-uniform flow is likely to occur. Assume any other data if not given.

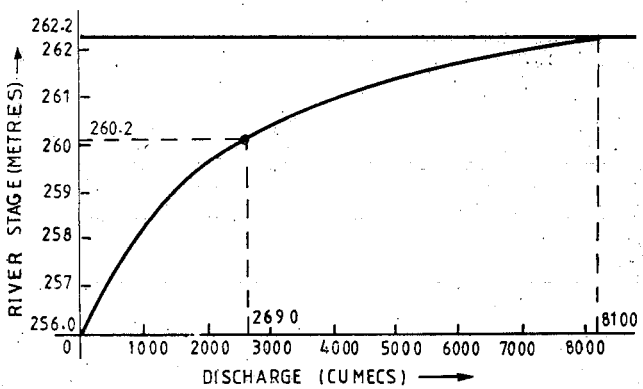


Fig. 11.19

Solution. Fixing the Crest Levels and Waterway

Crest Levels. The average bed level of the river is given to be 257.0 m, which may be taken as the crest level of the undersluices. The upstream floor of the undersluices may be kept the same, and thus there will be no raised crest in the undersluices. The crest level of other barrage bays may be kept 1.0 to 1.5 m higher than the crest level of undersluices. Let us keep it 1.3 m higher, *i.e.* say 257.0 + 1.3 = 258.3 m.

Waterway : The waterway, as per Lacey's wetted perimeter equation, is given by

$$P = 4.75\sqrt{Q}$$
$$= 4.75\sqrt{8100} = 4.75 \times 90 = \mathbf{427.5\ m.}$$

Now let us provide a waterway approximately equal to 1.2 P by trial, in such a way that approximately 15 to 20% of the discharge is passed through the undersluices and the total provided waterway should be able to pass the entire discharge.

Assume the waterway as below

(*a*) **Undersluice portion :**

5 bays of 15 m each	75 m
4 piers of 2.5 m each	10 m
Overall waterway	85 m

(*b*) **Other barrage bays portion**

27 bays of 12 m each	324.00 m
25 piers of 2 m each	50.00 m
Overall waterway	374.00 m

Assume a divide wall of 3.0 m thickness. Hence, total overall waterway provided between abutments = 85 + 374 + 3 = **462 m.**

Now, let us check whether the maximum flood can pass through this waterway with the maximum permissible afflux of 1.0 metre or not.

HFL before barrage construction = 262.2 m.

Permissible afflux = 1.0 m.

Now u/s HFL = d/s HFL + Afflux

d/s HFL = HFL before weir construction = 262.2 m

$\therefore$ u/s HFL = 262.2 + 1.0 = 263.2 m.

Average discharge intensity $= q = \dfrac{8{,}100}{462} = 17.6$

Scour depth $= R = 1.35\left(\dfrac{q^2}{f}\right)^{1/3}$

Assume Lacey's silt factor $f = 1.0$

$$R = 1.35\left[\frac{(17.6)^2}{1\cdot}\right]^{1/3} = 1.35\,(17.6)^{0.667} = 9.20 \text{ m.}$$

Velocity of approach $= V = \dfrac{q}{R} = \dfrac{17.6}{9.20} = 1.90$ m/sec.

Velocity Head $= \dfrac{V^2}{2g} = \dfrac{(1.90)^2}{2 \times 9.81} = 0.19$ m.

u/s TEL = u/s HFL + Velocity Head = 263.2 + 0.19 = 263.39 m.

Head (*i/c* velocity head) over the undersluice crest

= u/s TEL – Undersluice crest level = 263.39 – 257.0 = 6.39 m.

Head (*i/c* velocity head) over the crest of other barrage bays

= u/s TEL – Crest level of barrage bays = 263.39 – 258.3 = **5.09 m.**

Discharge passing through undersluices is given by the discharge formula for a broad crested weir; since the crest and u/s floor are at the same level, and the width of the crest is sufficient, it will behave as a broad crested weir.

$\therefore$ $Q_1 = 1.7\,(L - K\,n\,H)\,H^{3/2}$

where L is the clear waterway

Q_1 = Discharge through undersluices

$= 1.7\,[75 - 0.1 \times 10 \times 6.39]\,(6.39)^{3/2}$

$= 1.7\,[75 - 6.39]\,(6.39)^{3/2} = 1.7 \times 68.61 \times 16.1 =$ **1,880 cumecs.**

Let us keep the width of the crest of other barrage bays portion as 2.0 m. Since the head over the other barrage bays crest is 5.09 m., which is more than 1.5 times the width of the crest, it shall behave like a sharp crested weir. The discharge is then given by

$$Q = 1.84\,(L - 0.1\,nH)\,H^{3/2}$$

$\therefore$ Q_2 = Discharge through other barrage bays

$= 1.84\,(312.0 - 0.1 \times 52 \times 5.09)\,(5.09)^{3/2}$

$= 1.84\,(324.0 - 26.4)\,(5.09)^{3/2}$

$= 1.84 \times 297.6 \times 11.4 =$ **6, 260 cumecs**

Total discharge that can pass through the barrage

$= Q_1 + Q_2 = 1{,}880 + 6{,}260 = 8{,}140$ cumecs

$= 8{,}140$ cumecs $> 8{,}100$ cumecs.

Hence, the assumed waterway and crest levels are in order.

Actual overall waterway provided = 462 m against Lacey's wetted perimeter of 427.5 m.

$$\therefore \text{ Looseness factor} = \frac{462}{427.5} = \mathbf{1.08}$$

The design of undersluice section and that of other barrage bays section shall be carried out separately.

Design of Undersluice Portion

There are two major flow conditions : (*i*) When high flood is passing ; and (*ii*) When flow is at Pond Level (with all gates open). Let us calculate q and H_L for these two conditions.

(1) High flood condition

(*a*) *Assuming no concentration and retrogression*

u/s TEL = d/s HFL + Afflux + Velocity head = 262.2 + 1.0 + 0.19 = 263.39 m.

d/s TEL = d/s HFL + Velocity head = 262.2 + 0.19 = 262.39 m.

Head Loss $= H_L = 263.39 - 262.39 =$ **1.0 m**

Discharge intensity between piers $= q$

$= CH^{3/2} = 1.7\,(6.39)^{3/2} = 1.7 \times 16.1 = 27.4$ cumecs/metre.

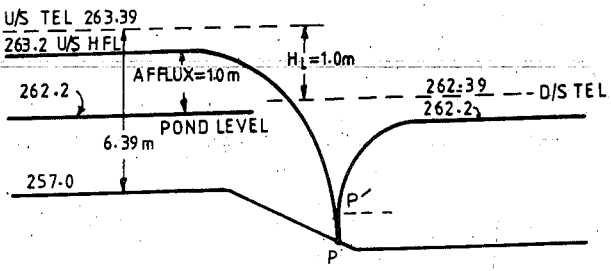


Fig. 11.20 (*a*) High Flood condition with no retrogression.

(b) High flood condition with 20% concentration and 0.5 m retrogression

The discharge intensity is increased by 20%. Therefore, new discharge intensity

$$= 1.2 \times 27.4 = 32.9 \text{ cumecs/metre}$$

New head (*i/c* velocity head) required for this discharge intensity to pass

$$= \left(\frac{32.9}{1.7}\right)^{2/3} = (19.5)^{0.667} = \mathbf{7.2\ m.}$$

The conditions are shown in Fig. 11.20 (*b*).

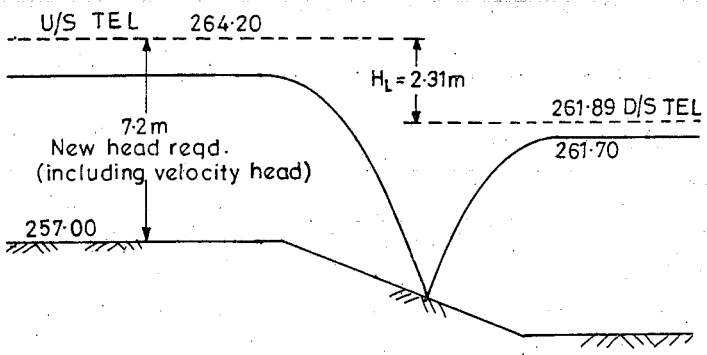


Fig. 11.20 (*b*) High flood condition with 20% concentration and 0.5 m retrogression.

u/s TEL = 257.0 + 7.2 = 264.2 m.

The d/s HFL is depressed by 0.5 m due to retrogression, *i.e.* it becomes 261.7 m

$\therefore$ d/s TEL = 261.7 + 0.19 = 261.89 m

$\therefore$ $H_L = 264.20 - 261.89 = \mathbf{2.31\ m.}$

(2) Pond Level Flow Condition

(a) With no concentration and retrogression

Pond Level (given) = 260.6 m.

Head over the crest of undersluices under this condition

$$= 260.6 - 257.0 = 3.6 \text{ m}$$

Head over the crest of other barrage bays

$$= 260.6 - 258.3 = 2.3 \text{ m.}$$

Neglecting velocity of approach for this flow condition, the total discharge passing down the barrage

$$Q = Q_1 + Q_2$$
$$= 1.7\,[75 - 0.1 \times 10 \times 3.6]\,(3.6)^{3/2} + 1.84\,[324 - 0.1 \times 52 \times 2.3]\,(2.3)^{3/2}$$
$$= 1.70 \times 71.4 \times 6.81 + 1.84 \times 312 \times 3.26$$
$$= 825 + 1865 = \mathbf{2690\ cumecs.}$$

Average discharge intensity $= \dfrac{2690}{462} = 5.82$ cumecs/metre

Normal scoured depth $= R = 1.35\left(\dfrac{q^2}{f}\right)^{1/3}$

$$= 1.35 \times \left[\frac{(5.82)^2}{1}\right]^{1/3} = 1.35 \times (5.82)^{0.667} = 4.45 \text{ m}$$

Velocity of approach

$$= V = \frac{q}{R} = \frac{5.82}{4.45} = 1.31 \text{ m/sec.}$$

$$\text{Velocity head} = \frac{V^2}{2g} = \frac{(1.31)^2}{2 \times 9.81} = 0.088 = \textbf{say 0.09 m.}$$

These conditions are shown in Fig. 11.21 (*a*).

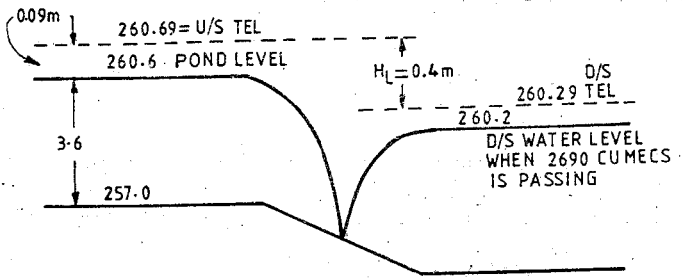


Fig. 11.21 (*a*) Pond level with no concentration and retrogression.

$\therefore$ d/s TEL = 260.6 + 0.09 = 260.69 m.

The d/s water level when a discharge of 2,690 cumecs is passing can be found from the Stage-Discharge curve of the river, given in Fig. 11.19. It is found to be 260.2 m.

$\therefore$ d/s TEL = 260.2 + 0.09 = 260.29 m.

$H_L = 260.69 - 260.29 = \textbf{0.40 m.}$

Discharge intensity between piers = $1.7 \times (3.6)^{3/2}$

$= 1.7 \times 6.81 = 11.54$ cumecs/metre.

(*b*) *Pond Level flow with 20% concentration and 0.5 m retrogression.*

New discharge intensity = $1.2 \times 11.54 = \textbf{13.86}$ cumecs/metre

$$\text{New head reqd. (}i/c\text{ velocity head) over the crest} = \left(\frac{13.86}{1.7}\right)^{2/3} = 4.06 \text{ m.}$$

U/s TEL = 257.0 + 4.06 = 261.06 m.

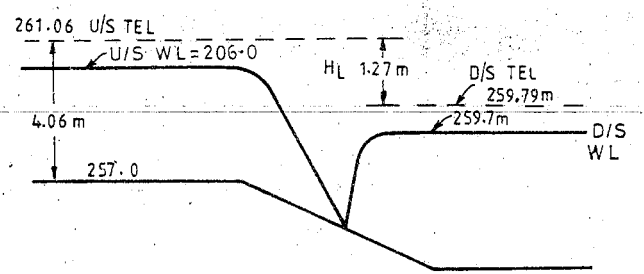


Fig. 11.21 (*b*) Pond level flow with 20% concentration and 0.5 m retrogression.

d/s HFL which was 260.2 m is depressed by 0.5 m. The new d/s HFL shall be 259.7 m.

$\therefore$ d/s TEL = 259.7 + 0.09 = 259.79 m.

$\therefore$ $H_L = 261.06 - 259.79 = \mathbf{1.27\ m.}$

The values of q, H_L, the water levels and energy levels for all these four cases are tabulated in Table 11.9.

Table 11.9. Undersluice Portion of Barrage

S. No.	Item	*High Flood Flow*		*Pond Level Flow*	
		condition 1(a) without concentration and retrogression	*condition 1(b) with concentration and retrogression*	*condition 2(a) without concentration and retrogression*	*condition 2(b) with concentration and retrogression*
(1)	(2)	(3)	(4)	(5)	(6)
1.	Discharge intensity (q) in cumecs/metre	27.4	32.7	11.54	13.86
2.	Upstream water level	263.2 m	263.2 m	260.60 m	260.6 m
3.	Downstream water level	262.2 m	261.7 m	260.20 m	259.7 m
4.	U/s TEL	263.39 m	264.20 m	260.69 m	261.06 m
5.	D/s TEL	262.39 m	261.89 m	260.29 m	259.79 m
6.	Head loss H_L	1.0 m	2.31 m	0.40 m	1.27 m
7.	E_{f2} (from Plate No. 10.1)	7.40 m	9.00 m	4.00 m	5.00 m
8.	Level at which jump will form, *i.e.* (d/s TEL $- E_{f2}$)	254.99 m	252.89 m	256.29 m	254.79 m
9.	$E_{f1} = E_{f2} + H_L$	8.40 m	11.31 m	4.40 m	6.27 m
10.	y_1 corresponding to E_{f1} (Plate No. 10.2)	2.4 m	2.5 m	1.65 m	1.30 m
11.	y_2 corresponding to E_{f2} (Plate No. 10.2)	6.40 m	8.0	3.30 m	4.5 m
12.	Length of concrete floor required = 5 ($y_2 - y_1$)	20.0 m	27.5 m	8.25 m	16.00 m
13.	Froude No. $F_1 = \frac{q}{\sqrt{g y_1^3}}$	2.36 m	2.66 m	1.74 m	2.97 m

All these four cases of jump formation are separately analysed. Using Blench curves, the values of $E_{f2} \cdot E_{f1}$, position of jump point P, y_2, y_1, etc. are all calculated as explained earlier and shown in Table 11.9. The table is self-explanatory. It can be seen from this Table 11.9, that the maximum value of 5 ($y_2 - y_1$) is 27.5 metres for the worst Case 1 (b), *i.e.* high flood flow with concentration and retrogression. Hence, we provide a *slightly conservative value of 29 m as the length of the downstream floor.* The lowest level at which jump will form, from this table, is 252.89 m [Case I (a)] and hence, we provide the downstream floor at a level of say *252.7 metres* (slightly lower than the calculated value of 252.89).

Hence, the d/s floor is provided at R.L. 252.7 m and equal to 29 metres in length, as shown in Fig. 11.22. The glacis is provided in 3 : 1 slope with a horizontal length of 12.9 metres, as shown in Fig. 11.22. Let us now calculate the total floor length required

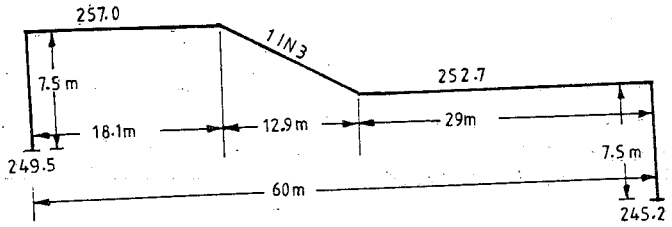


Fig. 11.22

from Khosla's Safe Exit Gradient Theory, so that the balance length of floor can be provided on the u/s side. Before this is calculated, we shall have to decide the depth of d/s cut off provided from scour considerations.

Depth of Sheet Pile Lines from Scour Considerations

Total discharge passing through undersluices = 1,880 cumecs (calculated earlier)

Overall waterway of undersliuces = 85 m.

Average discharge intensity $= \frac{1880}{85} = 22.3$ cumecs/metre.

Depth of scour

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

$$= 1.35\left[\left(\frac{22.3}{1}\right)^2\right]^{1/3} = 1.35(22.3)^{2/3} = 1.35 \times 8 = 10.8 \text{ m, say } 11 \text{ m.}$$

Let us provide a downstream cut-off at 1.5 R below the d/s water level (which is 261.7 m with retrogression). Hence, the R.L. of bottom of d/s cut-off

$$= 261.7 - 1.5 \times 11.0 = 261.7 - 16.5 = 245.2 \text{ m.}$$

Let us provide the d/s cut-off upto a bottom level of 245.2 m, i.e. a depth of 7.5 metres, as shown in Fig. 11.22.

U/s cut-off. Let us provide u/s cut-off at depth of 1.25 R (*i.e.* $1.25 \times 11.0 = 13.75$ m) from top of u/s water level.

Level of bottom of u/s cut-off $= 263.20 - 13.75 = 249.45$ m.

Let us, therefore, provide the u/s cut-off up to a bottom level of 249.5 m, *i.e.* 7.5 m deep, as shown in Fig. 11.22.

Total Floor length and Exit Gradient

Safe-exit gradient $(G_E) = \frac{1}{6}$

Maximum Static Head $(H) = 260.6 - 252.7 = \mathbf{7.9\ m}$

Depth of d/s cut-off $(d) = 252.7 - 245.2 = \mathbf{7.5\ m}$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$\therefore \quad \frac{1}{6} = \frac{7.9}{7.5} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{\pi\sqrt{\lambda}} = \frac{7.5}{7.9} \times \frac{1}{6} = 0.158$$

From Plate 11.2, value of α for a value of $\frac{1}{\pi\sqrt{\lambda}}$ as 0.158, comes out to be approximately 8.0.

$$\therefore \qquad b = \alpha \cdot d = 8 \times 7.5 = \mathbf{60\ m.}$$

Hence a balance of 18.1 m length can be provided as u/s floor length (shown in Fig. 11.22) so as to make the total floor length equal to 60.0 metres

Uplift Pressures

To calculate uplift pressures from Khosla's theory, let us first assume floor thickness at u/s cut-off as 1.0 m and at d/s cut-off as 1.5 m. Fig. 11.22 then becomes as shown in Fig. 11.23. No intermediate cut-off is necessary and hence not provided.

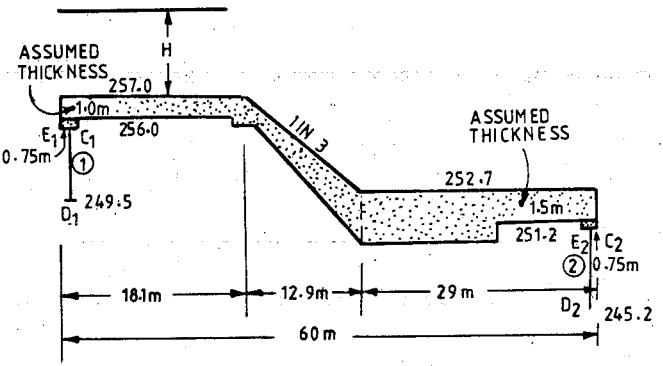


Fig. 11.23

Upstream Pile No. (1)

$$b = 60\ \text{m}$$

$$d = 7.5\ \text{m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$$

From Plate 11.1 (a)

$$\phi_{E_1} = 100\%$$

$$\phi_{C_1} = 100 - \phi_E = 100 - 32 = 68\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 22 = 78\%$$

D/s Pile Line

$$d = 7.5\ \text{m}$$

$$b = 60\ \text{m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{60} = 0.125$$

From Plate 11.1 (a)

$$\phi_{C_2} = 0\%$$

$$\phi_{E_2} = \phi_E = 32\%$$

$$\phi_{D_2} = \phi_D = 22\%$$

Let us correct these pressures

$$\phi_{C_1} = 68\%$$

$$\phi_{E_2} = 32\%$$

Corrections to ϕ_{C_1} :

(i) Effect of sheet pile No. (2) on pile No. (1) of depth d

$$\text{Correction} = 19 \times \sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

where D = depth of pile (2) below the point C_1, *i.e.* the point at which interference is desired

$= 256.0 - 245.2 = \mathbf{10.8\ m}$

$d = 256.0 - 249.5 = 6.5$ m

$b' = 58.5$ m ; $b = 60$ m

$$\text{Correction} = 19 \times \sqrt{\frac{10.8}{58.5}}\left[\frac{6.5+10.8}{60}\right] = 19 \times \frac{1}{2.33} \times \frac{17.3}{60} = 2.35\%\ (+\text{ve})$$

(ii) Correction for depth ;

$$\text{Correction} = \frac{78\% - 68\%}{257.0 - 249.5} \times 1.0 = \frac{10}{7.5} \times 1.0 = 1.33\%\ (+\text{ve})$$

ϕ_{C_1} (corrected) $= 68\% + 2.35\% + 1.33\% = \mathbf{71.68\%}$.

Corrections to ϕ_{E_2}

(i) Effect of sheet pile No. (1) on pile No. (2) of depth d

$$\text{Correction} = 19 \times \sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$$

where $D = 251.2 - 249.5 = 1.7$ m

$d = 251.2 - 245.2 = 6.0$ m

$b' = 58.5$ m

$b = 60$ m

$$\text{Correction} = 19 \times \sqrt{\frac{1.7}{58.5}}\left(\frac{6.0+1.7}{60.0}\right) = 19 \times \frac{1}{5.86} \times \frac{7.7}{60} = 0.42\%\ (-\text{ve})$$

Table 11.10

Condition of flow	*u/s water level in metres*	*d/s water level in metres*	*Head in metres (H)*	*Height/Elevation of Subsoil H.G. Line above Datum*					
				Upstream Pile Line No. (1)			*Downstream Pile Line No. 2*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *78%*	ϕ_{C_1} *= 71.68%*	ϕ_{E_2} *= 29.58%*	ϕ_{D_2} *22%*	ϕ_{C_2} *0%*
				7.9	6.17	5.66	2.33	1.74	0
No flow, maximum static Head	260.6	252.7 (No water *d/s*)	7.9	260.6	258.87	258.36	255.03	254.44	252.70
				1.5	1.17	1.07	0.44	0.33	0
(High Flood with concentration and retrogression)	263.2	261.7	1.5	263.2	262.87	262.77	262.14	262.03	261.7
				0.9	0.70	0.64	0.27	0.20	0
Flow at pond level (with concentration and retrogression)	260.6	259.7	0.9	260.6	260.40	260.34	259.97	259.9	259.7

(*ii*) Correction due to thickness of floor :

$$\text{Correction} = \left(\frac{32\% - 22\%}{252.7 - 245.2}\right) 1.5$$

$$= \frac{10}{7.5} \times 1.5 = 2.0\% \text{ (– ve)}$$

ϕ_{E_2} (Corrected) = 32% – 0.42% – 2.0% = **29.58%**

The levels of H.G. line at key points for various flow condition are tabulated in Table 11.10 as explained earlier.

We shall now determine the *hydraulic jump profiles* for the two flow conditions, *i.e.* High flood flow with concentration and retrogression, and pond level flow with concentration and retrogression.

(*a*) **Pre-jump profile** : It is worked out in Table 11.11 as explained earlier.

Table 11.11. Pre-jump Profile Calculations

Distance from the d/s end of the crest, i.e. the start of glacis, in metres	*Glacis level in metres*	*High Flood Flow q = 32.9 cumecs/metre*		*Pond Level Flow q = 13.86 cumecs/metre*	
		E_{f_1} *u/s TEL – Glacis Level, i.e. 264.20 – col. (2)*	y_1 *from Plate 10.2*	E_{f_1} *u/s TEL – Glacis Level, i.e. 261.06 – Col. (2)*	y_1 *from Plate 10.2*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	257.0	7.2	—	4.06	—
3	256.0	8.2	3.5	5.06	1.8
6	255.0	9.2	3.0	6.06	1.4
6.63	254.79 Point at which jump is formed at Pond Level	9.41	2.9	6.27	1.3
9.00	254.00	10.20	2.7		
12.33	252.89 Point at which jump is formed at high flood	11.31	2.5		

(*b*) **Post jump Profile**

From Table 11.9

Froude No. for high flood condition, F = 2.66, or $F^2 = 7.1$

Depth y_1 " " " = 2.5 m.

Froude No. for pond level flow condition = 2.97, or $F^2 = 8.8$

Depth y_1 " " " = 1.3 m.

From Plate 10.3 (*a*), the Table 11.12 is worked out :

Table 11.12. Post Jump Profile Calculations.

$\frac{x}{y_1}$ on Plate 10.3 (a)	High Flood Flow $F^2 = 7.1$, $y_1 = 2.5$ m			Pond Level Flow $F^2 = 8.8$, $y_1 = 1.3$ m		
	$\frac{y}{y_1}$ from Plate 10.3 (a)	y	x = Col. (1) × 2.5	$\frac{y}{y_1}$ from Plate 10.3 (a)	y	x = Col. (1) × 1.3
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.2	1.3	2.85	2.5	1.3	1.69	1.3
2.5	1.9	4.75	6.25	1.9	2.47	3.25
5.0	2.5	6.25	12.5	2.6	3.38	6.5
10.0	3.0	7.5	25.0	3.2	4.16	13.0

The Hydraulic jump profiles for these two flow conditions are plotted along with the H.G. lines for their respective conditions, as shown in Figs. 11.24 (*a*) and (*b*). From these

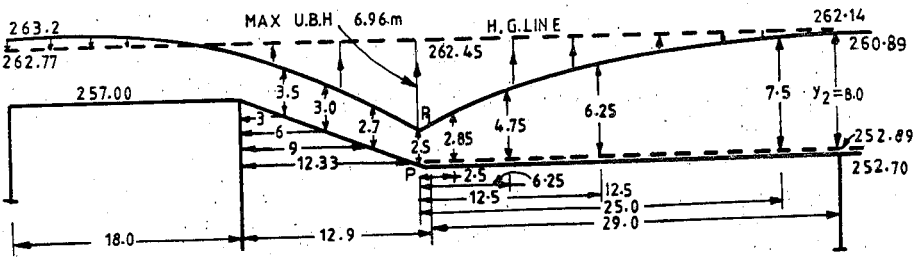


Fig. 11.24. (*a*). Unbalanced head in jump trough at high flood flow.

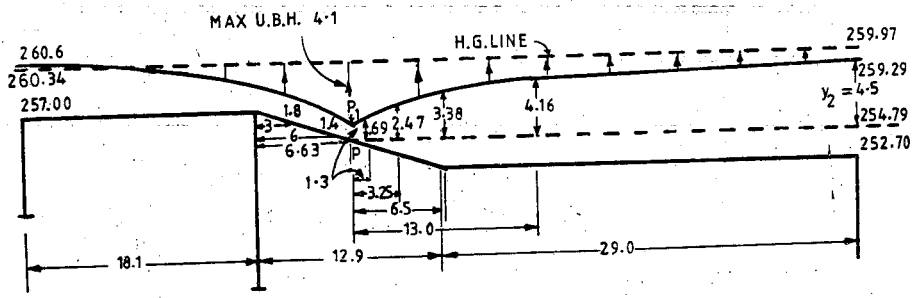


Fig. 11.24. (*b*). Unbalanced head in jump trough at pond level flow.

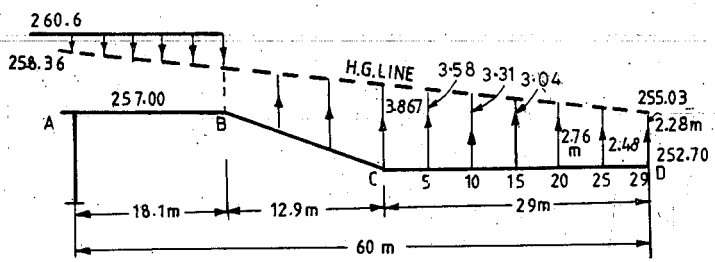


Fig. 11.24. (*c*). Unbalanced head in jump trough at static head at pond level.

figures, it is found that the maximum unbalanced head in the jump trough is equal to 6.96 m. The thickness for glacis shall, therefore, be designed for 2/3rd of this head, *i.e.* equal to 4.64 *metres,* or for the static condition head, whichever is greater. The H.G. line for the static head at pond level is also plotted in Fig. 11.24 (*c*).

It can be seen from the study of all these three figures, *i.e.* Fig. 11.24 (*a*), (*b*) and (*c*), that the dynamic action (*i.e.* flow) governs the floor thickness upto a length of about 5 m beyond the toe of the glacis, and the static head governs the rest of the downstream floor thickness.

Thickness required at point of jump

$$= \frac{4.64}{1.24} = 3.74 \text{ m}$$

Thickness required at 5 m beyond toe

$$= \frac{3.58}{1.24} = 2.88 \text{ m}$$

Thickness required at 15 m beyond toe

$$= \frac{3.04}{1.24} = 2.45 \text{ m}$$

Thickness required at 20 m beyond toe

$$= \frac{2.76}{1.24} = 2.23 \text{ m}$$

Thickness required at 25 m beyond toe

$$= \frac{2.48}{1.24} = 2.0 \text{ m}$$

Thickness required at 29 m beyond toe

$$= \frac{2.28}{1.24} = 1.84 \text{ m}$$

Provide floor thicknesses as shown in Fig. 11.25.

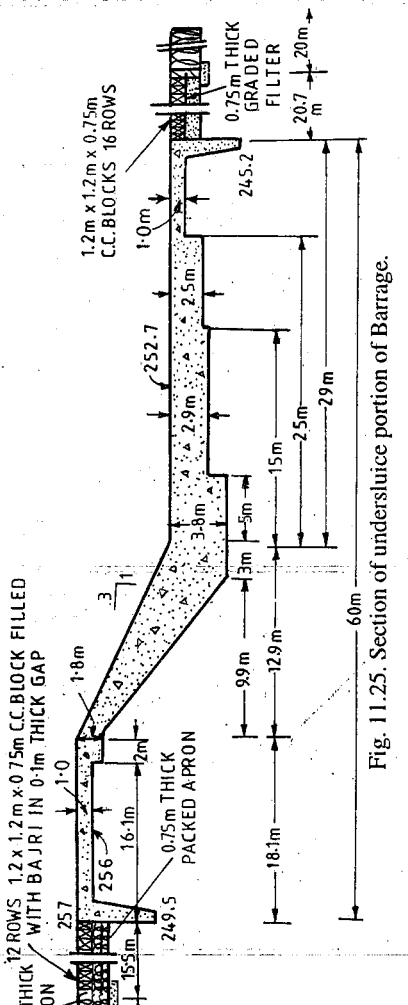


Fig. 11.25. Section of undersluice portion of Barrage.

Protection Works

Downstream Protection

Normal scour depth $= R = 11.0$ m

$$D = 2R - y = 2 \times 11.0 - (261.7 - 252.7)$$
$$= 22 - 9 = 13 \text{ m.}$$

Provide a launching apron equal 1.5*D*, *i.e.* say 20 m in length and of such a thickness (t) that its quantity is approximately 2.25 D cu. m/m.

$$\therefore \quad t = \frac{2.25 \times 13.0}{20} = 1.46 \text{ m ; } \textbf{Say 1.5 m thick.}$$

Let us provide c.c. blocks of size 1.2 m × 1.2 m × 0.75 m over an inverted filter of 0.75 m thickness for a length equal to say 1.5 *D*, *i.e.* approximately 20 m. 16 rows of c.c. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm gaps filled with bajri, shall be provided in length equal to 20.7 m, as shown in Fig. 11.25.

Upstream Protection

Normal scour depth $= R = 11.0$

Anticipated scour = $1.5\,R = 16.5$ m

$$D = 1.5\,R - y$$
$$= 1.5 \times 11.0 - (263.2 - 257.0) = 16.5 - 6.2 = 10.3 \text{ m}$$

Provide a launching apron of 1.5 m thickness in a length

$$= \frac{2.25 \times 10.3}{1.5} = 15.5 \text{ ; } \textbf{say 16 m in length.}$$

Let us provide c.c. blocks of size 1.2 m × 1.2 m × 0.75 m over packed apron of 0.75 m thickness for a length equal to say 1.5 *D*, *i.e.* 15.5 m in length. 12 rows of c.c. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm gaps filled with bajri shall be provided in length equal to 15.5 m, as shown in Fig. 11.25.

Design of Other Barrage Bays Portion

Here the crest level is 258.3 m. The four cases are analysed as given below :

Condition 1 (a) High flood flow with no concentration and retrogression.

u/s water level = 263.2 m
d/s water level = 262.2 m
u/s TEL = 263.39 m
d/s TEL = 262.39 m

$$H_L = 263.39 - 262.39 = 1.0 \text{ m.}$$

Head, including velocity head, over the crest

$$= 263.39 = 258.3 = 5.09 \text{ m}$$

Discharge intensity

$$q = 1.84\,(5.09)^{3/2} = 1.84 \times 11.4 = 21 \text{ cumecs/metre.}$$

Condition 1 (b) High flood flow with 20% concentration and 0.5 m retrogression

New discharge intensity

$$= 1.2 \times 21 = 25.2 \text{ cumecs/metre}$$

New head required, including velocity head, for this discharge to pass.

$$= \left[\frac{25.2}{1.84}\right]^{2/3} = (13.7)^{0.667} = 5.73 \text{ m.}$$

u/s TEL = 258.3 + 5.73 = 264.03 m
u/s TEL = 261.89 m

$$H_L = 264.03 - 261.82 = 2.14 \text{ metres.}$$

Condition 2 (a) : Pond level flow with no concentration and retrogression.

u/s W.L. = Pond Level = 260.6 m
d/s W.L. = 260.20 m
u/s TEL = 260.69 m
d/s TEL = 260.29 m

$$H_L = 260.69 - 260.29 = 0.4 \text{ m}$$

Head, including velocity head, over the crest = 260.69 – 258.3 = 2.39 m

Discharge intensity = $1.84\,(2.39)^{3/2} = 1.84 \times 3.69 = 6.78$ cumecs/metre.

Condition 2 (b) : Pond level flow with 20% concentration and 0.5 m retrogression.

New discharge intensity = $1.2 \times 6.78 = 8.13$ cumecs/metre.

New head required, including velocity head, for this discharge to pass

$$= \left[\frac{8.13}{1.84}\right]^{3/2} = (4.42)^{0.667} = 2.69 \text{ m}$$

u/s TEL = 258.3 + 2.69 = 260.99 m

d/s TEL = 260.29 – 0.5 = 259.79 m

H_L = 260.99 – 259.79 = **1.2 m.**

These values are tabulated in Table 11.13 and the table is completed as shown. The lowest level at which jump will form is found from this table as 254.39 m. Hence,

Table 11.13. Other Barrage Bays Portion

S. No.	Item	High Flood Flow		Pond level Flow	
		Condition 1 (a) without concentration and retrogression	Condition 1 (b) with 20% concentration and 0.5 m retrogression	Condition 2 (a) without concentration and retrogression	Condition 2 (b) with 20% concentration and 0.5 m retrogression
(1)	(2)	(3)	(4)	(5)	(6)
1.	Discharge intensity q in cumec/metre	21.0	25.2	6.78	8.13
2.	Upstream water level	263.2 m	263.2 m	260.6 m	260.6 m
3.	Downstream water level	262.2 m	262.2 m	260.2 m	259.7 m
4.	u/s TEL	263.39 m	264.03 m	260.69 m	260.99 m
5.	d/s TEL	262.39 m	261.89 m	260.29 m	259.79 m
6.	Head loss H_L	1.0 m	2.14 m	0.4 m	1.2 m
7.	E_{f1} from plate 10.1	6.3 m	7.5 m	3.0 m	3.7 m
8.	Level at which jump will form, *i.e.* d/s TEL – E_{f2}	256.09 m	**254.39 m***	257.29 m	256.09 m
9.	$E_{f1} = E_{f2} + H_L$	7.3 m	9.64 m	3.4 m	4.9 m
10.	y_1 corresponding to E_{f1} (plate 10.2)	2.1 m	1.9 m	1.1 m	0.9 m
11.	y_2 corresponding to E_{f2} (plate 10.2)	5.7 m	6.8 m	2.7 m	3.3 m
12.	Length of concrete floor required, *i.e.* 5 ($y_2 - y_1$)	5 (3.6) = 18.0 m	5 (4.9) = 24.5 m.	5 (1.6) = 8.0 m.	5 (2.4) = 12.0 m
13.	Froude No. $F_1 = \frac{q}{\sqrt{g y_1^3}}$	2.52	3.09	1.91	3.04 m

*Lowest point at which jump will form.

provide the downstream floor at level, say 254.2 m. (slightly lower than the calculated value of 254.39 m), and equal to *26 metres* in length. The glacis is provided in 3 : 1 slope with a horizontal length of 3 (258.3 – 254.2) = 3 × 4.1 = 12.3 *metres*. Crest width provided is 2 metres and upstream glacis in a slope of 1 : 1 with a horizontal length equal to 258.3 – 257.0 = 1.3 metres, as shown in Fig. 11.26.

Depth of Sheet Piles from Scour

Discharge passing = 6,260 cumecs

Overall waterway = 374 metres

Average discharge intensity $= \frac{6,260}{374} = 16.8$ cumecs/metre

$$R = 1.35\left[\frac{(16.8)^2}{1}\right]^{1/3} = 1.35 \times 6.6 = 9.07 \text{ m.}$$

Let us provide a downstream cutoff upto depth 1.5 R below the d/s water level which is 261.7 m with retrogression. Hence, the R.L. of bottom of d/s cut-off

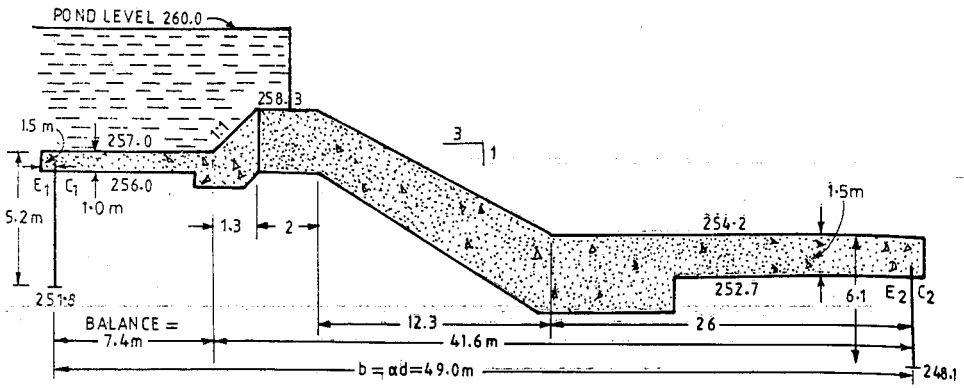


Fig. 11.26

$$= 261.7 - 1.5 \times 9.07$$
$$= 261.7 - 13.6 = 248.1 \text{ m.}$$

Let us provide d/s cut-off upto a bottom level of 248.1 m, *i.e.* depth of 6.1 m, as shown in Fig. 11.26.

U/s Cutoff. Let us provide 1.25 R, *i.e.*

$1.25 \times 9.07 = 11.4$ m for u/s cutoff.

$\therefore$ The level of bottom of u/s cutoff $= 263.2 - 11.4 = 251.8$ m.

Let us provide u/s cutoff upto a bottom level of 251.8 m, *i.e.* for a depth of 5.2 m, as shown in Fig. 11.26.

Total Floor Length and Exit Gradient

$$G_E = \frac{1}{6}.$$

Maximum Static Head $= H = 260.6 = 254.2 = 6.4$ m

Depth of d/s cutoff $= d = 254.2 - 248.1 = 6.1$ m

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

$\therefore$
$$\frac{1}{6} = \frac{6.4}{6.1} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

or
$$\frac{1}{\pi \cdot \sqrt{\lambda}} = \frac{1}{6} \times \frac{6.1}{6.4} = 0.159.$$

From Plate 11.2, value of α for a value of $\frac{1}{\pi\sqrt{\lambda}}$ as 0.159 comes out to be approximately 8.0.

$\therefore$ $b = \alpha d = 8 \times 6.1 = 48.8$ m ; say provide $b = 49$ m.

$\therefore$ Balance length of $49 - 1.3 - 2.0 - 12.3 - 26.0 = 49 - 41.6 = 7.4$ m is provided as upstream floor, as shown in Fig. 11.26.

Uplift Pressures

Let us assume floor thickness as 1.0 m at u/s cutoff end and 1.5 m at d/s cutoff end, as shown in Fig. 11.26.

Upstream Pile No. (1)

$$b = 49 \text{ m}$$

$$d = 5.2 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{5.2}{49.0} = 0.106.$$

From Plate 11.1 (*a*)

$$\phi_{E_1} = 100\%$$

$$\phi_{C_1} = 100 - \phi_E = 100 - 29 = 71\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 20 = 80\%$$

Downstream Pile No. (2)

$$d = 6.1 \text{ m}$$

$$b = 49.0 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{6.1}{49.0} = 0.125$$

From Plate 11.1 (*a*)

$$\phi_{C_2} = 0\%$$

$$\phi_{E_2} = \phi_E = 32\%$$

$$\phi_{D_2} = \phi_D = 22\%$$

Let us correct these pressures

$$\phi_{C_1} = 71\%$$

$$\phi_{E_2} = 32\%$$

Corrections to ϕ_{C_1}

(i) Effect of sheet pile No. (2) on pile No. (1) of depth d

$$\text{Correction} = 19 \times \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

where $d = 256.0 - 251.8 = 4.2$ m

$D = 256.0 - 248.1 = 7.9$ m

$b' = 47.5$ m

$b = 49.0$ m

$$\text{Correction} = 19 \sqrt{\frac{7.9}{47.5}} \left(\frac{4.2 + 7.9}{49} \right)$$

$$= 19 \times \frac{1}{2.45} \times \frac{12.1}{49} = 1.49\% \text{ (+ ve).}$$

(*ii*) Correction for depth of floor

$$\text{Correction} = \frac{80\% - 71\%}{257.0 - 251.8} \times 1.0 = \frac{9}{5.2} \times 1.0 = 1.73\% \text{ (+ ve)}$$

ϕ_{C_1} (*corrected*) $= 71\% + 1.49 + 1.73\% = 74.22\%$

Corrections to ϕ_{E_2}

(i) Effect of sheet pile No. (1) on sheet pile No. (2) of depth d

$$\text{Correction} = 19 \times \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b} \right)$$

where $d = 252.7 - 248.1 = 4.6$ m

$D = 252.7 - 251.8 = 0.9$ m

$b' = 47.5$ m

$b = 49.0$ m

$$\text{Correction} = 19 \times \sqrt{\frac{0.9}{47.5}} \left(\frac{4.6+0.9}{49} \right) = 0.29\% \text{ (– ve)}$$

(ii) Correction due to thickness of floor

$$\text{Correction} = \frac{32\% - 22\%}{254.2 - 248.1} \times 1.5 = 2.46\% \text{ (– ve)}$$

ϕ_{E_2} *(corrected)* $= 32 - 0.29 - 2.46 = 29.25\%$

The levels of H.G. line at key points for various flow conditions are tabulated in Table 11.14.

Table 11.14

Condition of flow	*U/s water level in metres*	*D/s water level in metres*	*Head in metres H*	*Height/Elevation of sub soil H.G. line above datum*					
				Upstream pile line No. (1)			*Downstream pile line No. (2)*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *80%*	ϕ_{C_1} *74.22%*	ϕ_{E_2} *29.25%*	ϕ_{D_2} *22%*	ϕ_{C_2} *0%*
				6.4	5.12	4.75	1.87	1.41	0
No flow, maximum static head	260.6	254.2 (No water)	6.4	260.6	259.32	258.95	256.07	255.61	254.2
				1.5	1.2	1.11	0.44	0.33	0
High flood flow with concentration and retrogression	263.2	261.7	1.5	263.2	262.90	262.81	262.14	262.03	261.7
				0.9	0.72	0.67	0.26	0.20	0
Flow at pond level with concentration and retrogression	260.6	259.7	0.9	260.60	260.42	260.37	259.96	259.90	259.7

We shall now determine the hydraulic jump profiles. For the two flow conditions.

Pre-jump Profile is worked out in Table 11.15.

Table 11.15. Pre-jump Profile Calculations

Distance from the start of 3 : 1 glacis	*Glacis level in metres*	*High Flood Flow with concentration and retrogression q = 25.2 cumecs/metre*		*Pond Level Flow with concentration and restogression q = 8.13 cumecs/metre*	
		E_{f_1} = *u/s TEL – glacis level i.e. 264.03 —col. (2)*	y_1 *From Plate 10.2*	E_{f_1} = *u/s TEL – glacis level i.e. 260.99 – col. (2)*	y_1 *From Plate 10.2*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
0	258.3	5.73	—	2.69	—
3	257.3	6.73	3.0	3.69	1.15
6	256.3	7.73	2.25	4.69	1.00
6.63	256.09	7.94	2.2	4.90	0.90
	Point at which jump forms for Pond Level flow				
9.0	255.3	8.73	2.1	—	—
11.73	254.39	9.64	1.9	—	—
	Point at which jump forms for High Flood flow				

Post Jump Profile. If (x, y) are the ordinates of any point taking the point of jump P' as the origin, the profile can be worked out using plate 10.3 (b) as shown in Table 11.16. This profile can also be worked out by using plate 10.3 (a) as was done in the design of undersluice portion (Table 11.12). We have used both these plates just to explain their respective use.

Table 11.16. Post Jump Profile Calculations

Values of x where x is the horizontal distance from the point of jump	*High Flood Flow* $y_2 - y_1 = 4.9$, $F_1 = 3.09$			*Pond Level Flow* $y_2 - y_1 = 2.4$, $F_1 = 3.04$		
	$\frac{x}{y_2 - y_1}$	$\frac{y}{y_2 - y_1}$ *From Plate 10.3 (b)*	y	$\frac{x}{y_2 - y_1}$	$\frac{y}{y_2 - y_1}$ *From Plate 10.3 (b)*	y
2 m	0.41	0.16	0.79	0.83	0.33	0.79
4 m	0.81	0.32	1.57	1.67	0.51	1.22
6 m	1.20	0.38	1.86	2.50	0.68	1.63
8 m	1.61	0.50	2.45	3.33	0.77	1.85
10 m	2.01	0.58	2.84	4.17	0.88	2.11
15 m	3.06	0.76	3.73	6.25	0.98	2.35
20 m	4.08	0.86	4.21	8.25	1.00	2.40
25 m	5.10	0.95	4.67	10.4	—	—

Hydraulic jump profile for two flow conditions, their H.G. lines and the uplift pressure diagrams are plotted in Fig. 11.27 (a) and (b). The H.G. line and the uplift pressure diagram for maximum static head is also plotted in Fig. 11.27 (c). By studying

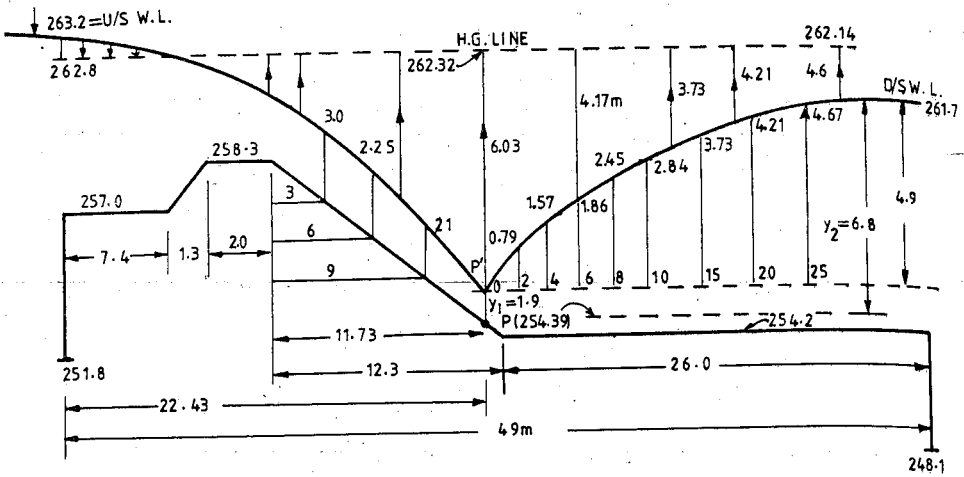


Fig. 11.27. (*a*). Uplift pressures in the jump trough for high flood flow.

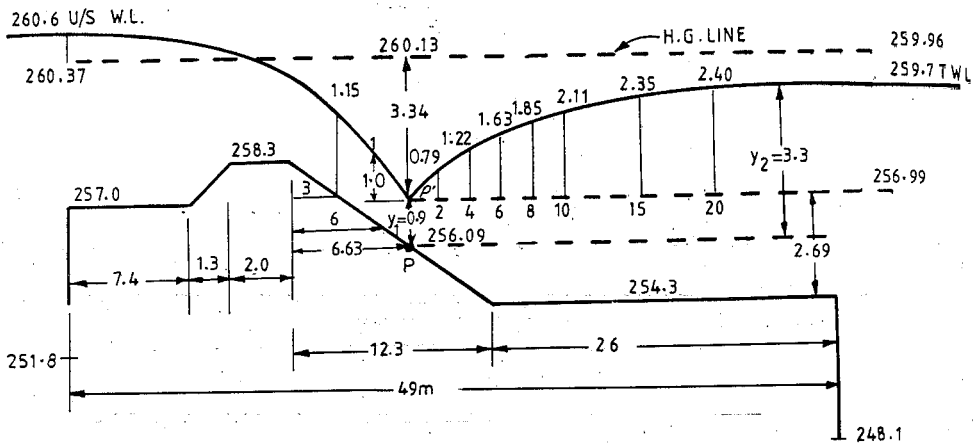


Fig. 11.27. (*b*). Uplift pressures in the jump trough for pond level flow.

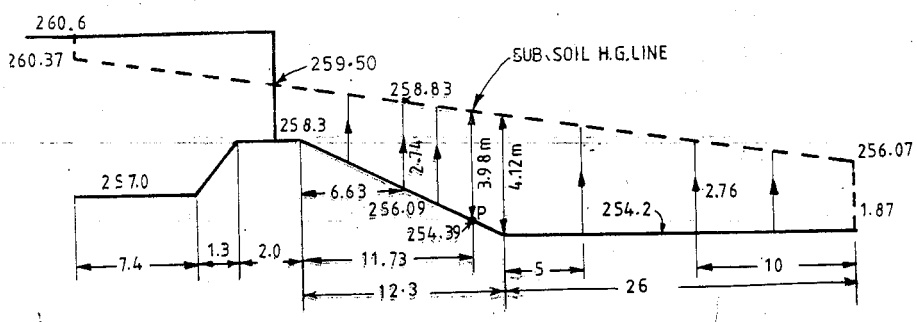


Fig. 11.27. (*c*). Uplift pressures in the jump trough for max. static head at pond level.

these diagrams, it is found that the maximum designed head for dynamic action is $\frac{2}{3} \times 6.03 = 4.02$ m [Fig. 11.27 (*a*)]. The maximum ordinate for uplift in static head condition = 4.12 m [Fig. 11.27 (*c*)]. Hence, static head condition becomes the governing factor. So minimum thickness required.

$$= \frac{4.12}{1.24} = 3.32 \text{ m.}$$

So we *provide 3.5 m.*

For the rest of the downstream floor beyond 5 m of the toe of the glacis, the thickness is decided by dividing the uplift ordinates of Fig. 11.27 (*c*) by 1.24; and by making suitable adjustments, the depths are provided as shown in Fig. 11.28.

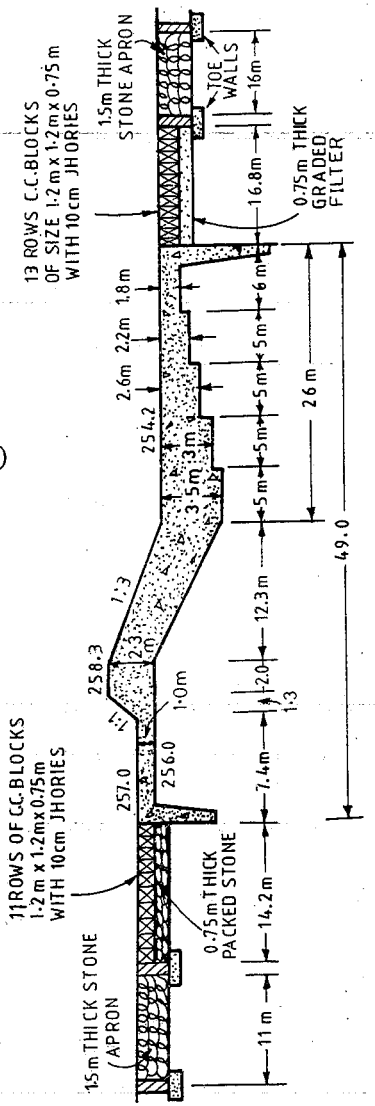


Fig. 11.28. Section of other Barrage Bays

Protection Works

(*i*) *Downstream Protection*

Normal scour depth

$$R = 9.07 \text{ m}$$

$$D = 2R - y = 2 \times 9.07 - (261.7 - 254.2)$$
$$= 18.14 - 7.5 = 10.64 \text{ m}$$

Provide a launching apron equal to 1.5 D, *i.e.* say 16 m in length and of thickness say 1.5 m.

Let us provide C.C. blocks of size 1.2 m × 1.2 m × 0.75 m over a graded filter of 0.75 m thickness for a length equal to 1.5 D, *i.e.* approximately 16 m. 13 rows of C.C. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm gaps filled with bajri shall hence, be provided in length equal to 16.8 metres.

(*ii*) *Upstream Protection*

Normal scour depth

$$= R = 9.07 \text{ m}$$

$$D = 1.5\,R - y$$
$$= 1.5 \times 9.07 - (263.2 - 257.0)$$
$$= 13.6 - 6.2 = 7.4 \text{ m.}$$

Provide a launching apron of thickness 1.5 m in a length

$$= \frac{2.25 \times 7.4}{1.5} = 11.1 \text{ m.} = \textbf{Say 11 m.}$$

Let us provide C.C. blocks of size 1.2 m × 1.2 m × 0.75 m over packed stone of 0.75 m thickness for a length equal to say 1.5 D *i.e.* 13.6 m. Hence, provide 11 rows of C.C. blocks of size 1.2 m × 1.2 m × 0.75 m having 10 cm jhories (*i.e.* gaps filled with bajri), in a length equal to 14.2 metres, as shown in Fig. 11.28.

Canal Head Regulator Design

Example 11.7. *Design a suitable head regulator for the barrage designed in example 11.6. The following data for the off-taking canal are also given :*

Full supply discharge of canal = *180 cumecs*
Anticipated maximum full supply level of canal = *260.2 m*
Bed level of canal = *257.2 m*
Safe exit gradient for canal bed material = *1/5.*

Solution. Students are advised to go through the theory for Canal Head Regulator and its design, etc. given in Chapter 9, before solving this problem.

The crest level of the canal head regulator is kept 1.2 to 1.5 m higher than the crest level of the undersluices. The crest level of undersluices (from previous example) = 257.0 m

Pond Level (from previous example, = 260.6 m
u/s H.F.L. (from previous example) = 263.2 m
Let us keep the **crest level** of regulator (*i.e.* sill level) = 257.0 + 1.5 = **258.5 m.**

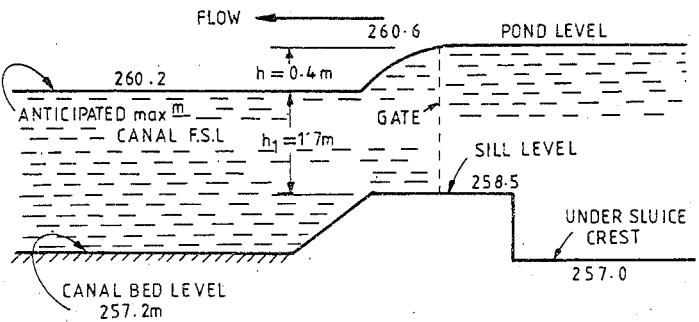


Fig. 11.29.

Let u/s now fix the waterway for the regulator, such that full supply discharge of 180 cumecs can pass through it.

Discharge Q through the regulator is given by

$$\frac{2}{3} C_{d_1} \cdot \sqrt{2g} \cdot B \cdot (h + h_v)^{3/2} + C_{d_2} B \cdot h_1 \sqrt{2g\,(h + h_v)}$$

Taking $C_{d_1} = 0.577$

and $C_{d_2} = 0.80$

and neglecting velocity head h_v, we get

$$Q = \frac{2}{3} \times 0.577 \cdot \sqrt{2g} \cdot B \,.\, h^{3/2} + 0.80 \,.\, B \,.\, h_1 \sqrt{2gh}.$$

Here in this example, we have

$h = 0.4$ m
$h_1 = 1.7$ m
$Q = 180$ cumecs

$$\therefore \quad 180 = \frac{2}{3} 0.577 \sqrt{19.62}. \quad B\,(0.4)^{3/2} + 0.80\,B \,.\, (1.7) \sqrt{19.62 \times 0.4}$$

$$= B\,(1.69 \times 0.253 + 1.36 \times 2.8) = 4.237\,\mathbf{B}$$

or $$B = \frac{180}{4.237} = \mathbf{42.5\ m}$$

Provide 6 bays of 7.5 m each giving a clear waterway of 45 m. Provide 5 piers of 1.5 m each ; Overall waterway of the regulator = $45 + 5 \times 1.5 = 52.5$ m.

Hydraulic Calculations for Various Flow Conditions

(*i*) *Full supply discharge passing down the regulator during high flood.* When u/s water level is 263.2 m (*i.e.* High flood), water shall pass over the regulator and the gated opening provided between the sill level and pond level shall have to be adjusted by partially opening this gate, as shown in Fig. 11.30.

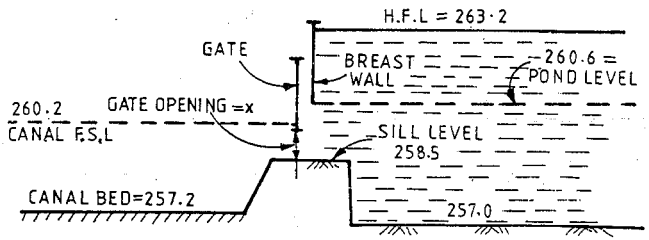


Fig. 11.30

Let the gate opening be x metres. The discharge can then be calculated by submerged orifice formula. *i.e.*

$$Q = C_d \,.\, A \,.\, \sqrt{2gh}$$

where $C_d = 0.62$

$A = B \,.\, x = 45 \,.\, x\ \text{m}^2$

$Q = 180$ cumecs.

h = head causing flow = $263.2 - 260.2 = 3.0$ m.

$\because$ $180 = 0.62\,(45x)\,\sqrt{2 \times 9.81 \times 3}$

or $180 = 0.62 \times 45 \times 7.67\,x$

or $$x = \frac{180}{0.62 \times 45 \times 7.67} = \mathbf{0.84\ m.}$$

Velocity of flow through opening

$$= \frac{180}{45 \times 0.84} = 4.76\ \text{m/sec}$$

Loss of head at entry

$$= 0.5\,\frac{V^2}{2g} = \frac{0.5 \times (4.76)^2}{2 \times 9.81} = 0.58\ \text{metres}$$

TEL just u/s of gate

$$= 263.2 + 0.19 = 263.39\ \text{m}$$

[$\because$ Vel. head is calculated in previous example = 0.19 m]

TEL just downstream of gate = $263.39 - 0.58 = 262.81$ m.

Downstream water level = 260.2 m.

Head loss $= H_L = 262.81 - 260.2 = \mathbf{2.61\ m.}$

Discharge intensity $q = \frac{180}{45} = 4.4$ cumecs/metre.

(ii) Full supply discharge passing down the regulator at pond level.

$H_L = 260.6 - 260.2 = 0.4$ m

$q = 4.0$ cumecs/metre.

Hydraulic jump calculations for the two flow conditions are tabulated in Table 11.17.

Table 11.17

S.No.	Item	*High flood flow condition*	*Pond level flow condition*
1	Discharge intensity q in cumecs/m	4.0	4.0
2	Upstream water level	263.2 m	260.6 m
3	Downstream water level	260.2 m	260.2 m
4	u/s TEL	262.81 m	260.6 m
5	d/s TEL	260.2 m	260.2 m
6	Head loss H_L	2.61 m	0.4 m
7	E_{f_2} (from Plate 10.1)	2.75 m	2.05 m
8	Level at which jump will form *i.e.* d/s TEL – E_{f_2}	257.45 m	258.15 m
9	$E_{f_1} = E_{f_2} + H_L$	5.36 m	2.54 m
10	y_1 corresponding to E_{f_1} (Plate 10.2)	0.45 m	0.68 m
11	y_2 corresponding to E_{f_2} (Plate 10.2)	2.67 m	1.90 m
12	Length of concrete floor required = 5 $(y_2 - y_1)$	$5 \times 2.22 = 11.1$	$5 \times 1.22 = 6.1$ m
13	Froude No. $F_1 = \frac{q}{\sqrt{g y_1^3}}$		

Provide downstream floor at R.L. 257.2 metres with a horizontal length of 13 metres.

Depth of Sheet Piles from Scour Considerations

Discharge intensity $q = 4.0$ cumecs/metre

D/s Sheet Pile. Depth of scour 'R' $= 1.35 \cdot \left(\frac{q^2}{f}\right)^{1/3}$

$= 1.35 \cdot \left(\frac{4^2}{1}\right)^{1/3} = 1.35 \times 2.52 = 3.4$

Let us provide a d/s cutoff upto 1.5 R *i.e.* 5.1 m below the d/s water level which is 260.2 m.

$\therefore$ R.L. of bottom of d/s cutoff $= 260.2 - 5.1 = 255.1$ m.

This value gives only $257.2 - 255.1 = 2.1$ m deep cutoff, which is small. *So let us provide d/s cutoff up to a bottom level of 253.0 m i.e. 4.2 m deep below the d/s floor level.*

U/s Sheet Pile. Provide upstream sheet pile line down to the elevation 249.5 m (*i.e.* the same as that of the undersluices).

Total Floor Length and Exit Gradient

The worst condition of flow occurs when the maximum flood is passing in the river (u/s water level 263.2 m) and there is no water in the canal (*i.e.* the canal is completely closed). The bed level of d/s floor = 257.2 m.

Maximum static head under this condition

$$= H = 263.2 - 257.2 = 6.0 \text{ m}$$

Depth of d/s cutoff = $(d) = 4.2$ m.

$$G_E = \frac{1}{5} \text{ (Given)}$$

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$\therefore \quad \frac{1}{5} = \frac{6.0}{4.2} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or $$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{4.2}{6.0} = \mathbf{0.140}$$

From Plate 11.2

$$\alpha = 9$$

$$\therefore \quad b = \alpha \cdot d = 9 \times 4.2 = 37.8 \text{ m}$$

Adopt total floor length = 38 metres.

The floor length shall be provided as below and as shown in Fig. 11.31.

D/s horizontal length	= 13.0 m
D/s glacis length with 3 : 1 slope	= 4.2 m.
Crest width	= 2.0 m
Balance provided as upstream floor	= 18.8 m
	Total = 38.0 m

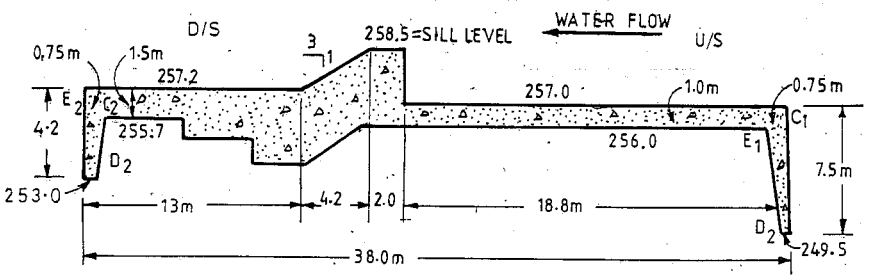


Fig. 11.31

Uplift Pressures

Let us assume 1.0 m floor thickness on u/s and 1.5 m on downstream, as shown in Fig. 11.31.

Upstream pile No. 1.

$$b = 38 \text{ m}$$

$$d = 7.5 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{7.5}{38} = 0.197.$$

From Plate 11.1,

$$\phi_{E_1} = 100\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 27\% = 73\%$$

$$\phi_{C_1} = 100 - \phi_E = 100 - 39\% = 61\%$$

Downstream pile No. (2)

$$b = 38 \text{ m}$$

$$d = 4.2$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{42.}{38} = 0.11.$$

From Plate 11.1

$$\phi_{E_2} = \phi_E = 30\%$$

$$\phi_{D_2} = \phi_D = 20\%$$

$$\phi_{C_2} = 0\%$$

These pressures are to be corrected as follows :

Corrections to ϕ_{C_1}

(i) Correction due to the effect of sheet pile No. (2) on pile No. (1) of depth d.

$$\text{Correction} = 19 \cdot \sqrt{\frac{D}{b'}} \cdot \left[\frac{d+D}{b}\right]$$

where $d = 256.0 - 249.5 = 6.5$ m

$D = 256.0 - 253.0 = 3.0$ m

$b' = 36.5$ m

$b = 38$ m.

$$\text{Correction} = 19 \cdot \sqrt{\frac{3.0}{36.5}} \left(\frac{6.5+30}{38}\right) = 1.36\% \text{ (+ ve)}$$

(ii) Correction due to floor thickness

$$\text{Correction} = \frac{73\% - 61\%}{7.5} \times 1.0 = 1.6\% \text{ (+ ve)}$$

$$\phi_{C_1} \textit{ (corrected)} = 61\% + 1.36\% + 1.6\% = \mathbf{63.96\%}$$

Corrections to ϕ_{E_2}

$$\phi_{E_2} = 30\%$$

(i) Correction due to the effect of sheet pile No. (1) on sheet pile No. (2) of depth d.

$$\text{Correction} = 19 \sqrt{\frac{D}{b'}} \left(\frac{d+D}{b}\right)$$

where $d = 255.7 - 253.0 = 2.7$ m

$D = 255.7 - 249.5 = 6.2$ m

$b' = 36.5$ m

$b = 38$ m.

$$\text{Correction} = 19 \cdot \sqrt{\frac{6.2}{36.5}} \left(\frac{2.7+6.2}{38}\right) = 1.84\% \text{ (– ve).}$$

(ii) Correction for floor thickness

$$\text{Correction} = \frac{30\% - 20\%}{257.2 - 253.0} \times 1.5 = 3.57\% \text{ (– ve)}$$

$$\phi_{E_2} \text{ (corrected)} = 30 - 1.84 - 3.57 = 24.59\%$$

The levels of the H.G. lines at key points for different flow conditions are tabulated in Table 11.18.

Table 11.18

Condition of flow	*U/s water level in metres*	*D/s water level in metres*	*Head in metres (H)*	*Height/elevation of sub-soil H.G. line above datum*					
				Upstream pile line No. (1)			*Downstream pile line No. (2)*		
				ϕ_{E_1} *100%*	ϕ_{D_1} *73%*	ϕ_{C_1} *63.96%*	ϕ_{E_2} *24.59%*	ϕ_{D_2} *20.0%*	ϕ_{C_2} *0%*
No flow, maximum static head	263.2	257.2 No water in canal	6.0	6.0 263.2	4.38 261.58	3.84 261.04	1.48 258.68	1.20 258.40	0 257.2
High flood on barrage and full supply discharge through canal head regulator	263.2	260.2 (Canal FSL)	3.0	3.0 263.2	2.19 262.39	1.92 262.12	0.74 260.94	0.60 260.8	0 260.2
Flow at pond level	260.6	260.2 (Canal FSL)	0.4	0.4 260.6	0.29 260.49	0.26 260.46	0.10 260.30	0.08 260.28	0 260.2

Floor Thicknesses

The maximum static head on the floor occurs for the worst condition *i.e.* when high flood is passing down the barrage but the canal is empty and regulator gates are closed.

∴ The maximum static head = 263.2 – 257.2 = 6.0 metres

The subsoil H.G. line shall be drawn for the maximum static head only (Fig. 11.32) since the floor thicknesses are governed by this critical condition.

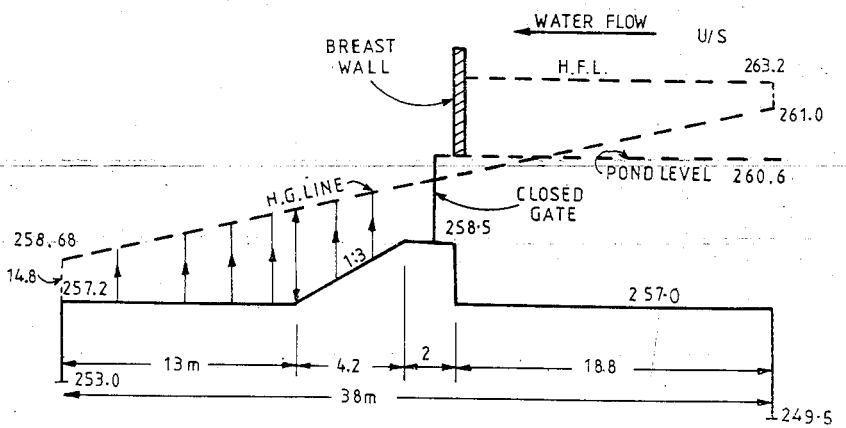


Fig. 11.32. Max. Static Head condition.

Thickness reqd. at d/s end of floor $= \frac{1.48}{1.24} = 1.19$ m.

At 4 m from end, d/s unbalanced head $= 1.48 + \frac{2.36}{38} \times 4 = 1.73$ m.

Thickness reqd. $= \frac{1.73}{1.24} = 1.4$ m.

Provide 1.5 metres thickness in this 4 m reach

At 8 m from d/s end, unbalanced head $= 1.48 + \frac{2.36}{38} \times 8$
$= 1.98$ m

Thickness reqd. $= \frac{1.98}{1.24} = 1.61$ m.

Provide 1.8 m thickness in this portion

At 13 m from d/s end, unbalanced head

$$= 1.48 + \frac{2.36}{38} \times 13 = 2.29 \text{ m}$$

Thickness required $= \frac{2.29}{1.24} = 1.85$ m

Provide 2 m thickness in this portion.

The provided floor thicknesses are shown in Fig. 11.33.

Protection works

(*i*) *U/s Protection.* It shall be equal to what was provided in upstream of under-sluices

(*ii*) *D/s Protection*

$R = 3.4$ m (calculatedearlier)

Anticipated scour $= 2R = 2 \times 3.4 = 6.8$ m.

The level of d/s scour hole

= d/s water level $- 2R$

$= 260.2 - 6.8 = 253.4$ m

Scour depth (D) below d/s floor

$= 257.2 - 253.4 = \mathbf{3.8\ m}$

Let us provide a launching apron in a length equal to 1.5 D (say 5.7 m) and of thickness

$$= \frac{2.25 \times 3.8}{5.7} = 1.5 \text{ m.}$$

Use 6 m length of 1.5 m thick launching apron.

Let us use 1.3 m × 1.3 m × 0.75 m C.C. block in a length equal to app. 1.5 D, *i.e.* 5.7 m. Hence, use 4 rows of 1.3 m × 1.3 m × 0.75 m C.C. blocks with 10 cm jhories in between filled with bajri, in a total length of 5.5 m.

The protection works are shown in Fig. 11.33.

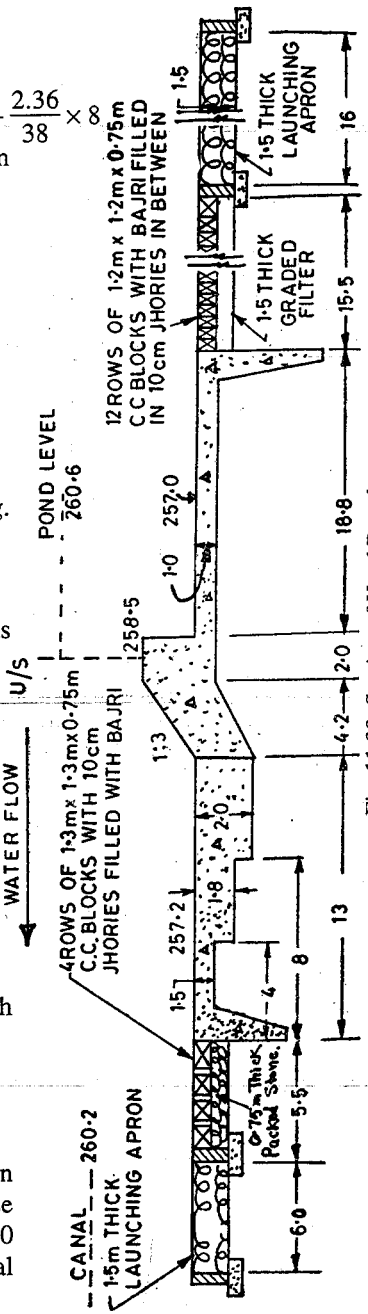


Fig. 11.33. Section of Head Regulator.

CERTAIN IMPORTANT INDIAN BARRAGES

11.7. Data Pertaining to Certain Important Barrages of India

The data pertaining to the location, purpose, flood discharge, and other characteristics of certain important barrages/weirs constructed on Indian rivers, are given below :

11.7.1. Durgapur Barrage. Durgapur barrage (Fig. 11.34) is situated at **Durgapur in Burdwan Distt., West Bengal State** (on Damodar river). It helps in serving *irrigation, industrial, water supply,* and *navigation* needs. The work on this barrage was started in

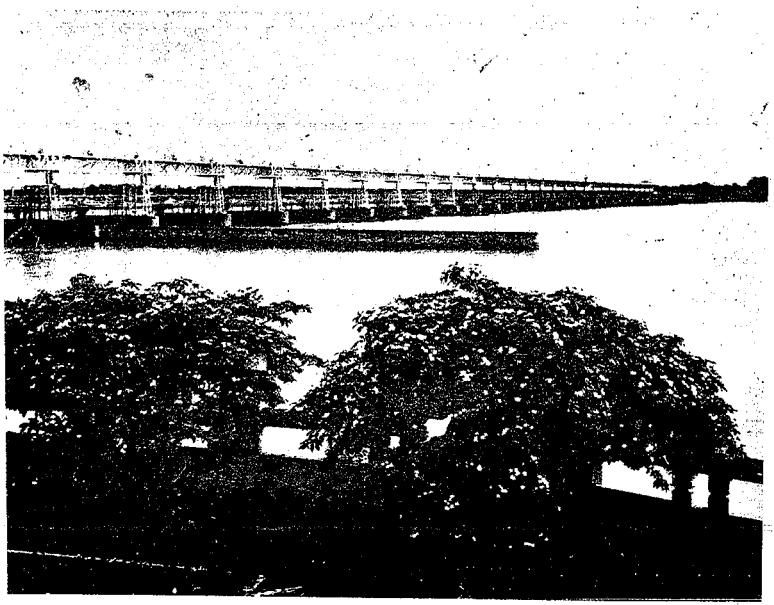


Fig. 11.34. (*a*). Photoview of Durgapur Barrage.

the year 1952, and completed in 1955. The *design flood discharge* for the barrage site is 15,576 cumecs (5.5 lakh cusecs). Other details of the barrage and its canal head regulator are given below.

Hydraulic Particulars of Durgapur Barrage

Pond Level	...211.5 m
Width of river	...2,000 m
Length of barrage/anicut	...692.2 m
No. of under-sluice bays	...10
No. of barrage bays	...24
Width of under-sluice bays	...18 m each
Width of barrage-bays	...18 m each
Thickness of intermediate piers	...2.13 m

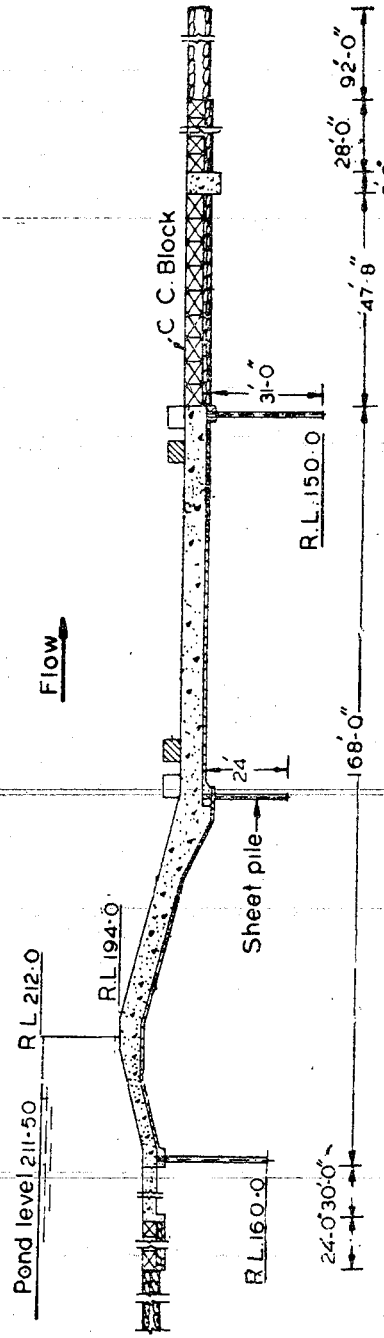


Fig. 11.34. (*b*). Section through under-sluices of Durgapur Barrage.

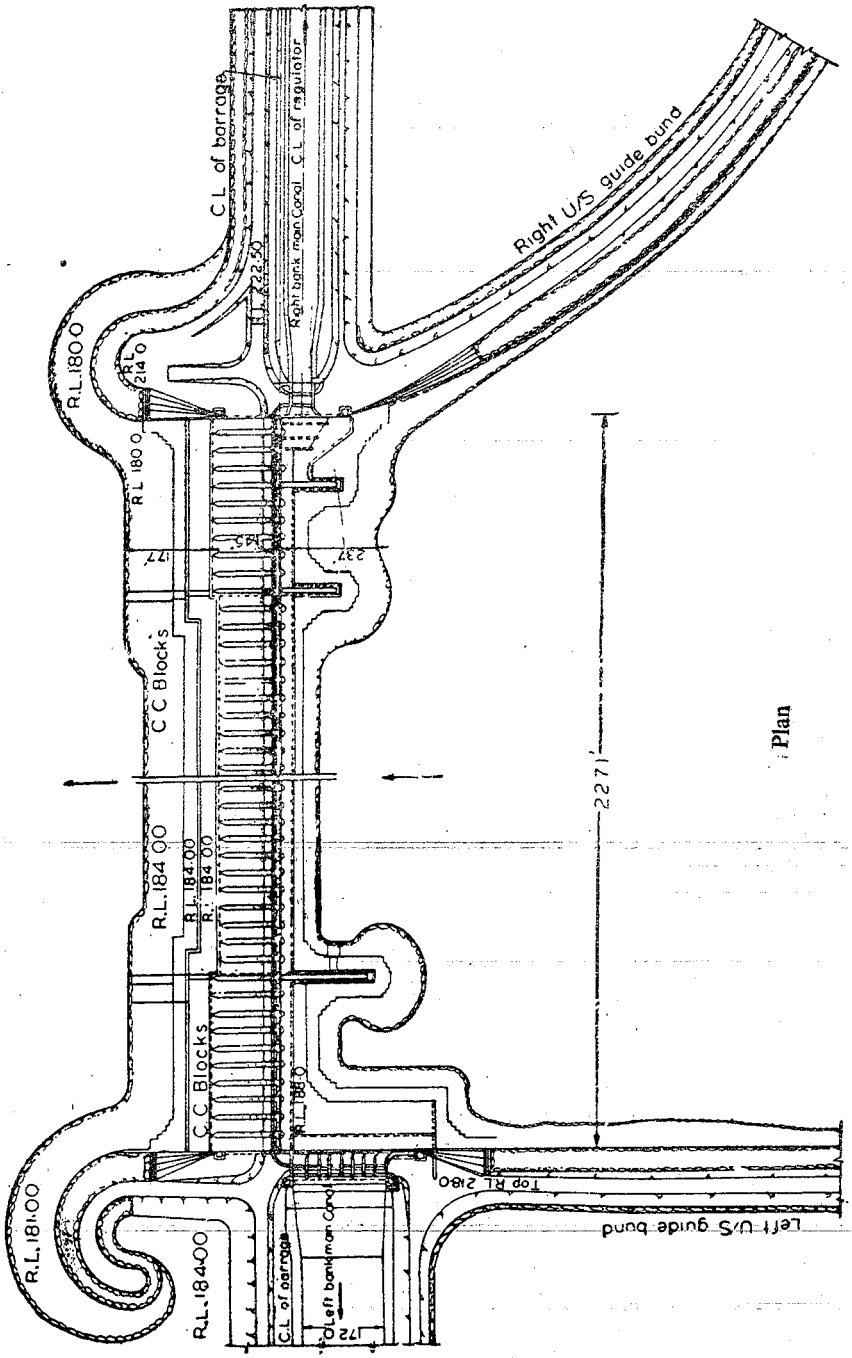


Fig. 11.34. (c). Plan of Durgapur Barrage.

Gates. (*a*) *Number* ...24 in weir bays
...10 in undersluice (barrage) bays.

(*b*) *Size* Barrage ...4.87 m× 18 m
Left undersluice ...5.38 m × 18 m
Right undersluice ...5.79 m × 18 m.

Sediment excluding devices ...Silt excluder and under-sluices.

Length of upstream divide wall ...80.47 m.

Canal Head Regulator Details

	Left	*Right*
Width of head regulator	59.44 m	13.71 m
No. of bays and their width	8 of 6.09 m each	2 of 6.09 m each
Thickness of piers	2.44 m	1.52 m
Orientation with respect to barrage axis	Perpendicular to the barrage axis	
Location of head regulator from barrage axis	16 m from G.L. of barrage axis	
Energy dissipating devices below head regulator	c.c. blocks and boulder pitching apron	
Max. discharge of canal	260.18 cumecs	64.31 cumecs

11.7.2. Bhimgoda Barrage. Bhimgoda barrage is a new barrage, the construction of which, has recently been completed, across the holy Ganges at Hardwar in U.P. This barrage is a replacement of the old Bhimgoda Weir (Photoview in Fig. 11.35). The old weir is presently being dismantled.

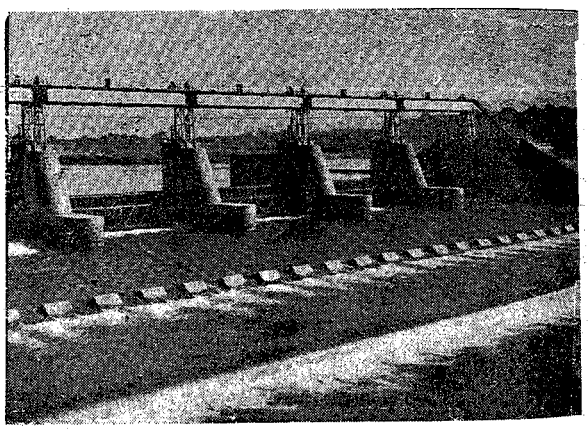


Fig. 11.35. Photoview of the Old Bhimgoda Weir (constructed in the year 1920).

The layout plan of the new barrage w.r. to the old weir is shown in Fig. 11.36 (*a*). Sections of barrage bays and undersluice bays are also shown in Fig. 11.36 (*b*) and 11.36 (*c*) ; respectively.

This headwork serves the irrigation needs of the adjoining areas, and is designed for a flood discharge of 19300 cumecs (6.8 lakh cusecs).

Other particulars of this headworks are given below.

Hydraulic Particulars of Bhimgoda Barrage

Pond level ...290.2 m (min.) and
293.7 m (max.)

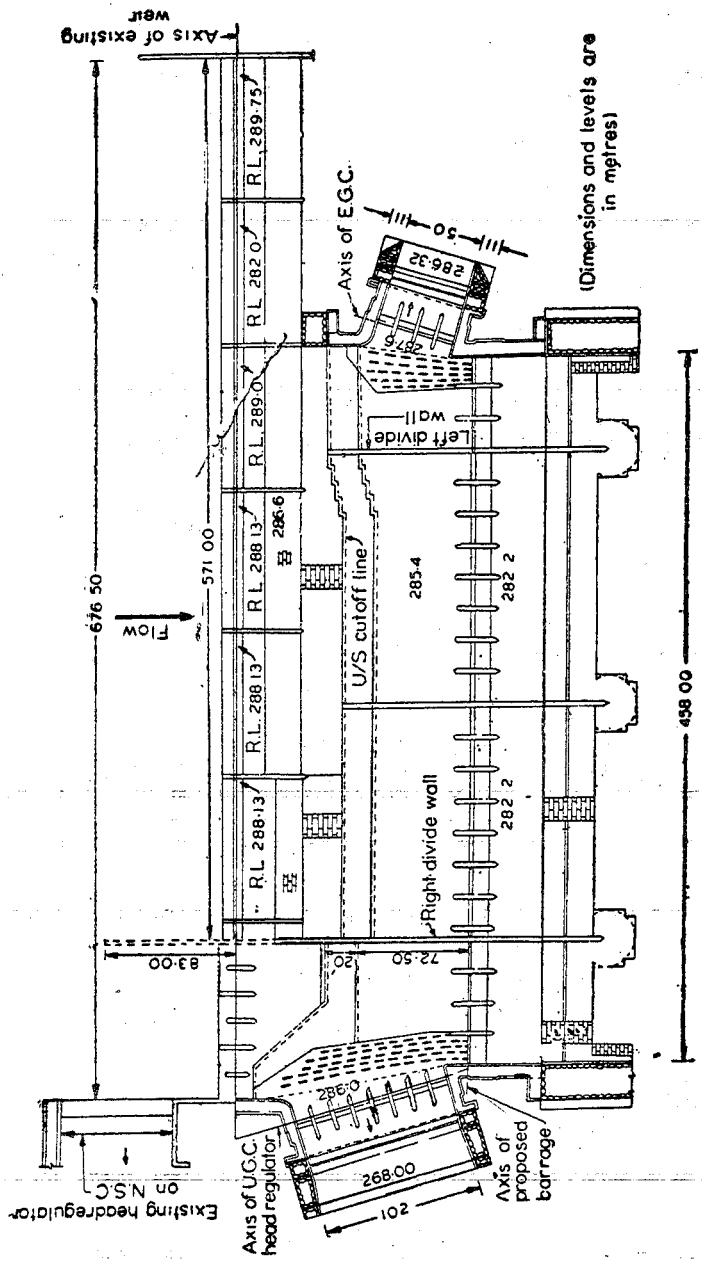


Fig. 11.36. (*a*) Layout Plan of New Bhimgoda Headworks (Barrage) on Ganga river in U.P.

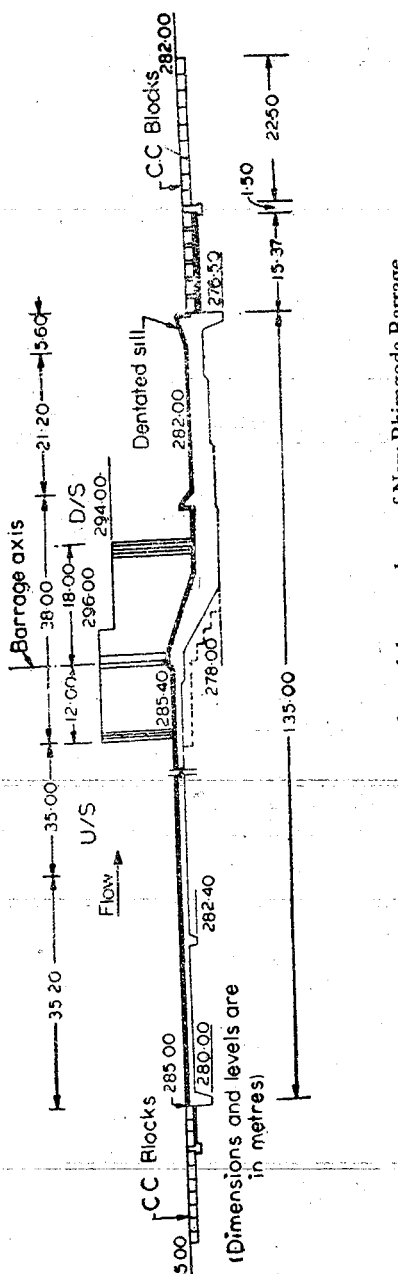


Fig. 11.36. (*b*). Section through barrage bays of New Bhimgoda Barrage

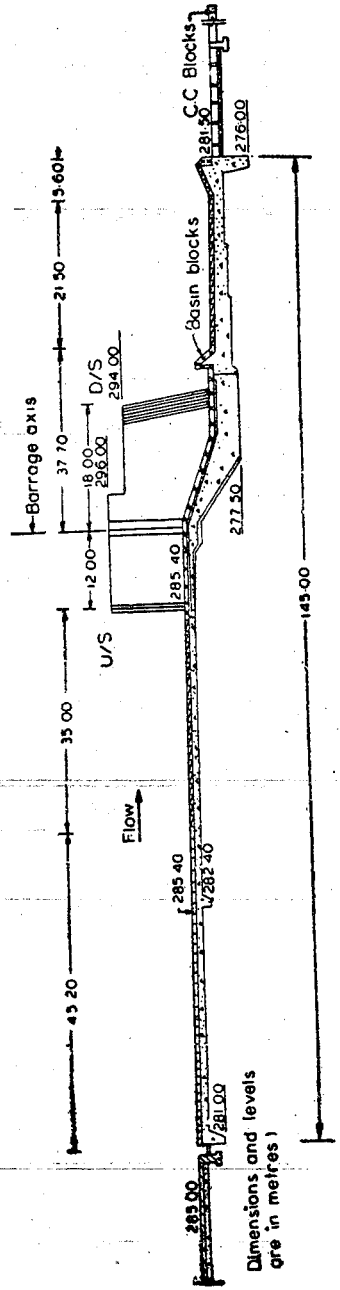


Fig. 11.36. (*c*). Section through undersluice bays of New Bhimgoda Barrage.

Width of river	...675 m
Length of barrage	...455 m
No. of under-sluice bays	...7
No. of barrage bays	...15
Width of under-sluice bays	...18 m each
Width of barrage bays	...18 m each
Thickness of intermediate piers	...2.5 m each
Gates	
(*a*) Number	...22
(*b*) Size	...18 m × 7.8 m for barrage bays
	...18 m × 8.4 m for under-sluices
Sediment Excluding Devices	...Silt excluder and silt ejector
Length of u/s divide wall	...110 m
Energy Dissipation Arrangements	...Basin blocks and dentated sills

Canal Head Regulator Details

	Right Bank Regulator	*Left Bank Regulator*
Width of head regulator	102 m	50 m
No. of bays and their width	8 of 11 m each	4 of 11 m each
Thickness of piers	2 m	
Orientation with respect to barrage axis	107°	
Location of head regulator from barrage axis	66 m (app.)	40 m (app.)
Energy Dissipation method below regulator	Basin blocks	
Max. discharge of canal	410 cumecs	164 cumecs

11.7.3. New Okhla Barrage. This is a new barrage constructed on Yamuna river, at New Delhi, about 3 km downstream of the existing Old Okhla weir. It aims to serve *irrigation* and *water supply* needs through the New Agra Canal. The design flood discharge is 8495 cumecs. (3 lakh cusecs).

Other particulars of this barrage (Fig. 11.37) and its canal head regulator are given below :

Hydraulic Particulars of New Okhla Barrage

Pond level	...201.35 m

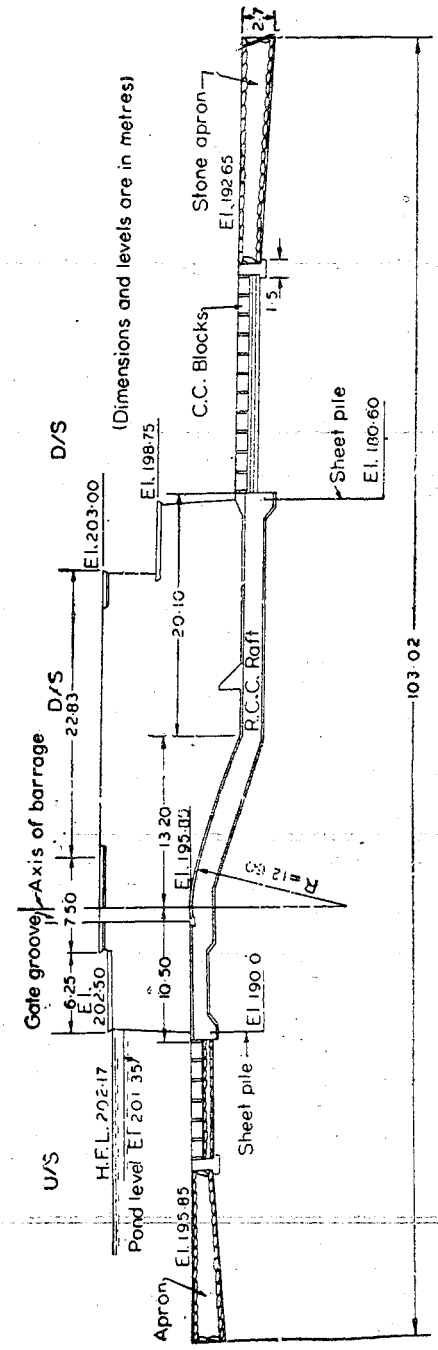


Fig. 11.37. New Okhla Barrage at New Delhi on Yamuna river.

Width of river ...445.73 m

Length of barrage ...552 m

No. of under-sluice bays ...5

No. of barrage bays ...22

Width of under-sluice bays ...18.3 m each

Width of barrage bays ...18.3 m each

Thickness of intermediate pier ...2.13 m

Gates

(*a*) *Number* ...27

(*b*) *Size* For under-sluices : 5 of 18.3 m × 6 m each

For others : 22 of 18.3 m × 5.1 m each

Sediment Excluding Devices ...Silt excluder

Length of u/s Divide wall ...85.42 m

Energy Dissipation Arrangements ...Baffle blocks and Dentated sills

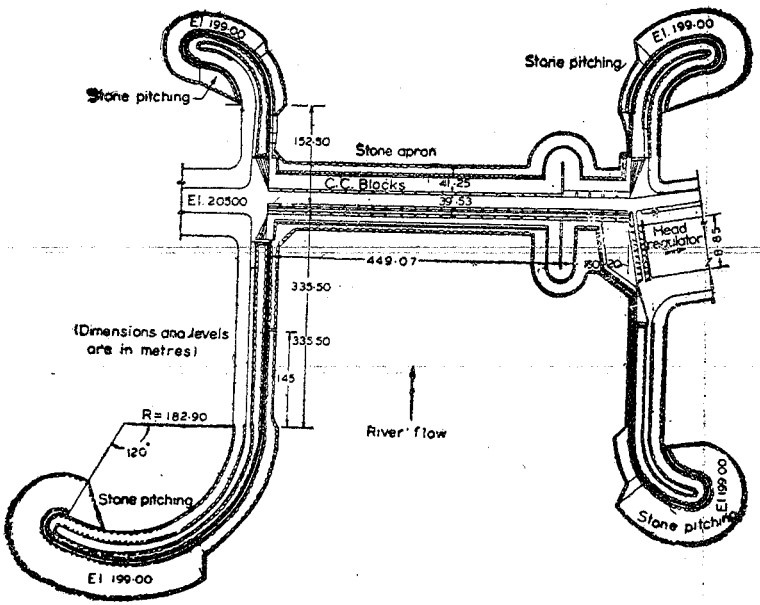


Fig. 11.37 (*c*) Layout Plan of New Okhla Barrage at New Delhi on Yamuna river.

Canal Head Regulator Details

Width of head regulator ...80.85 m

No. of bays and their width ...9 of 7.65 m each

Thickness of piers ...1.5 m each

Orientation with respect to barrage axis ...100°

Location of Head regulator from barrage axis	...22.25 m u/s on right bank,
Energy dissipation arrangements below regulator	...Baffle blocks and end sills
Max. discharge of canal	...242.4 cumecs.

11.7.4. Grand Anicut. The famous old *Grand Anicut* (Fig. 11.38) is located at 16 km from Tiruchirapalli (Tamil Nadu State) on Cauvery river, and is situated 193 km downstream of the famous Mettur dam (Coloured Photo Fig. 11.39) near Salem.

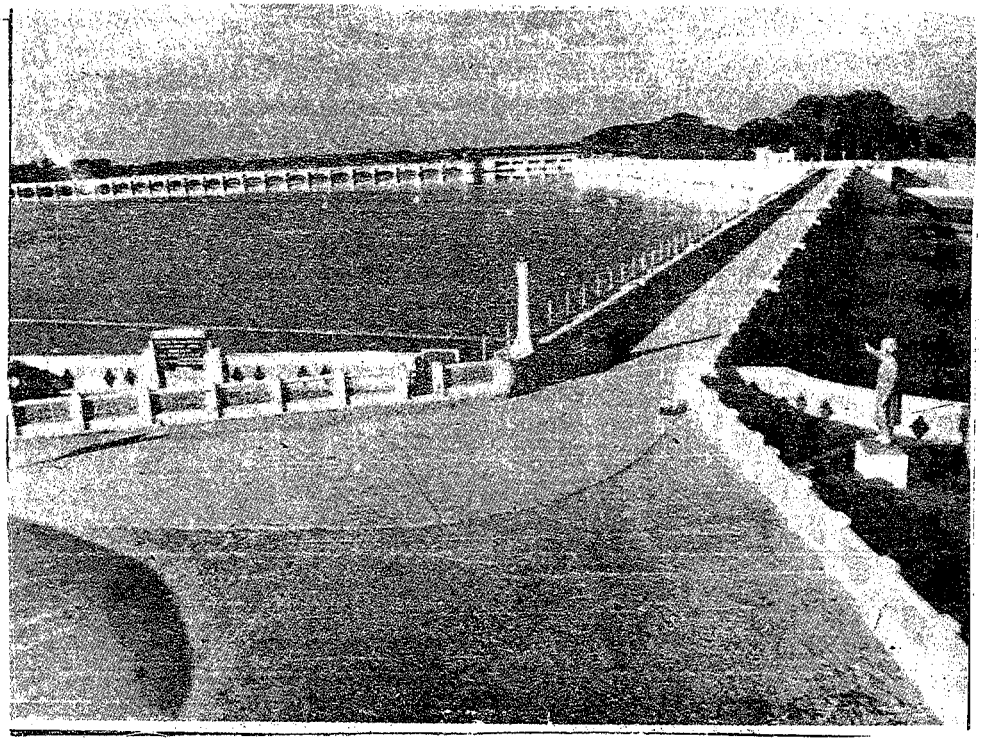
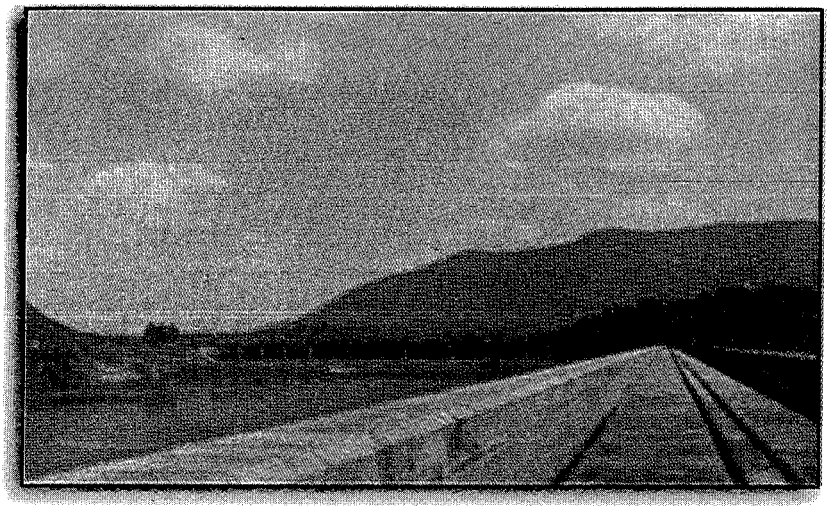


Fig. 11.38 (*a*) Photoview (close) of Grand Anicut.

It serves the *irrigation* needs of the adjoining areas. The design flood discharge is 5094 cumecs (1.8 lakh cusecs). It was constructed in the year 200 A.D. Other particulars of this project are given below :

Hydraulic Particulars of the Grand Anicut

Pond Level	...61.567 m
Width of river	...419 m
Length of anicut	...329 m
No. of Under-sluice bays	...5
No. of Weir bays	...30
Width of Under-sluice bays	...6.1 m each
Width of weir bays	...9.75 m each



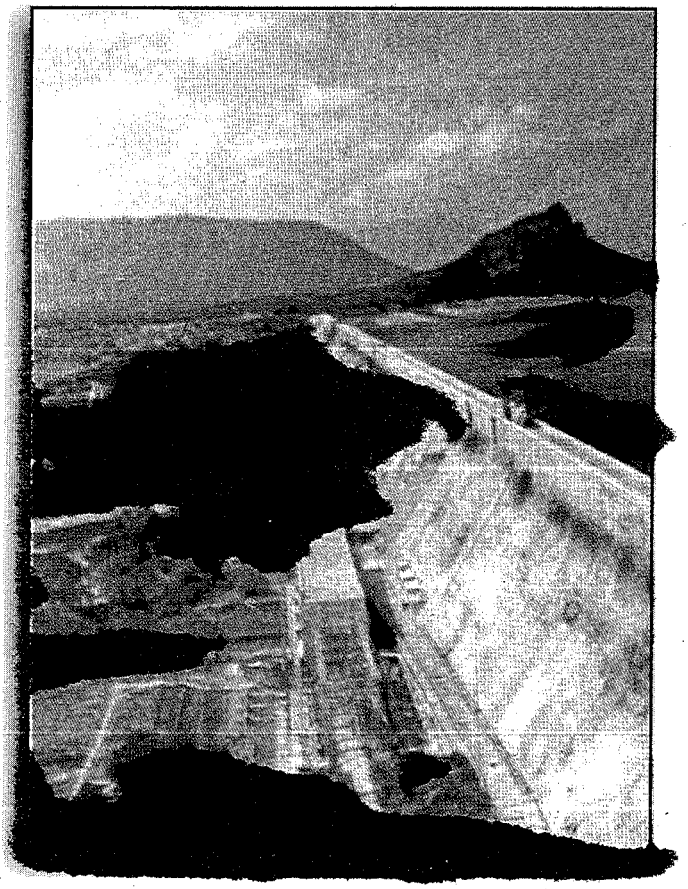


Fig. 11.39. Two views of Mettur dam.

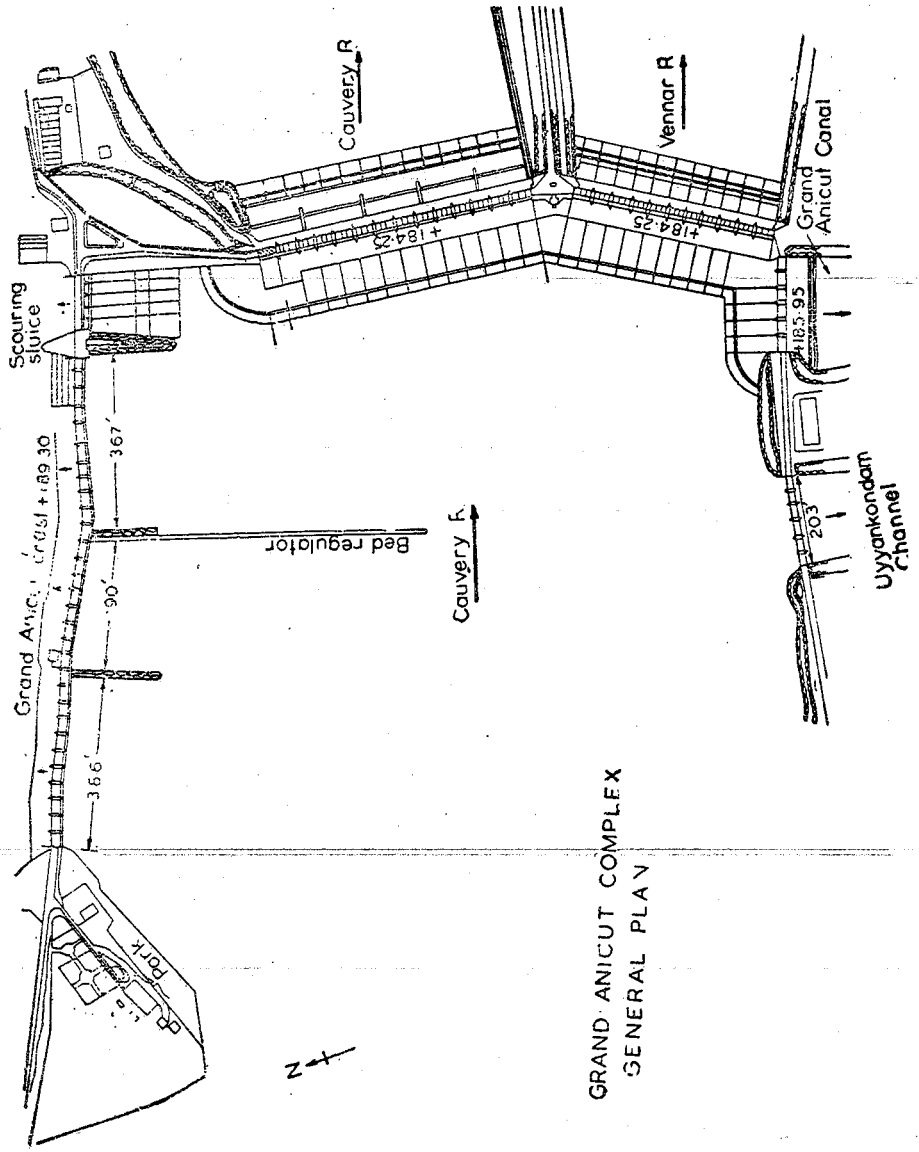


Fig. 11.38 (*b*) General Plan of Anicut Complex (Grand Anicut).

Thickness of intermediate pier	...1.22 m (weir) and 1.83 m (under-sluices)
Gates	
(*a*) Number	...30
(*b*) Size	...9.75 m × 1.52 m
Sediment Excluding Devices	...Under-sluices
Energy Dissipation Arrangements	...Stepped apron

Canal Head Regulator Details

	Cauvery	*Vennar*	*Grand Anicut canal*
Width of head regulator	183 m	135 m	61 m
No. of bays and their width	42 of 3.05 × 2.74 m each	33 of 3.05 × 3.45 m each	6 of 9.14 × 1.6 m each
Thickness of Piers	4.27 m	3.35 m	1.22 m
Orientation with respect to barrage axis	90°	90°	Parallel
Location of head regulator from barrage axis	61 m	61 m	419 m on the right side
Energy Dissipation Devices below regulator.	Baffle blocks		
Max. Discharge of canal	441 cumecs	376 cumecs	116 cumecs

11.7.5. Prakasm Barrage. The famous Prakasm barrage (Fig. 11.40) is located at Vijaywada (in Andhra Pradesh State) on Krishna river. It basically serves the *irrigation*

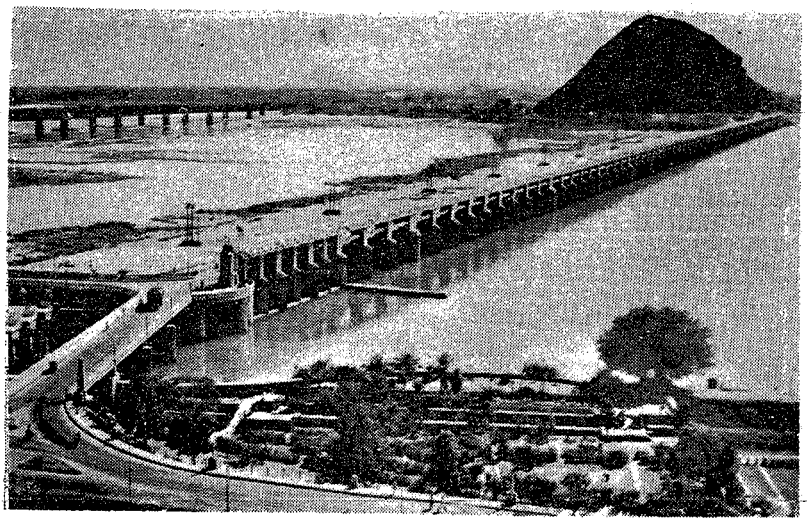


Fig. 11.40. (*a*) Photoview of Prakasm Barrage.

needs of the adjoining areas. The Design flood discharge is as high as 33,984 cumecs (12 lakh cusecs). Its construction started in 1954 and got completed in 1957. Other particulars and details of this barrage are given below :

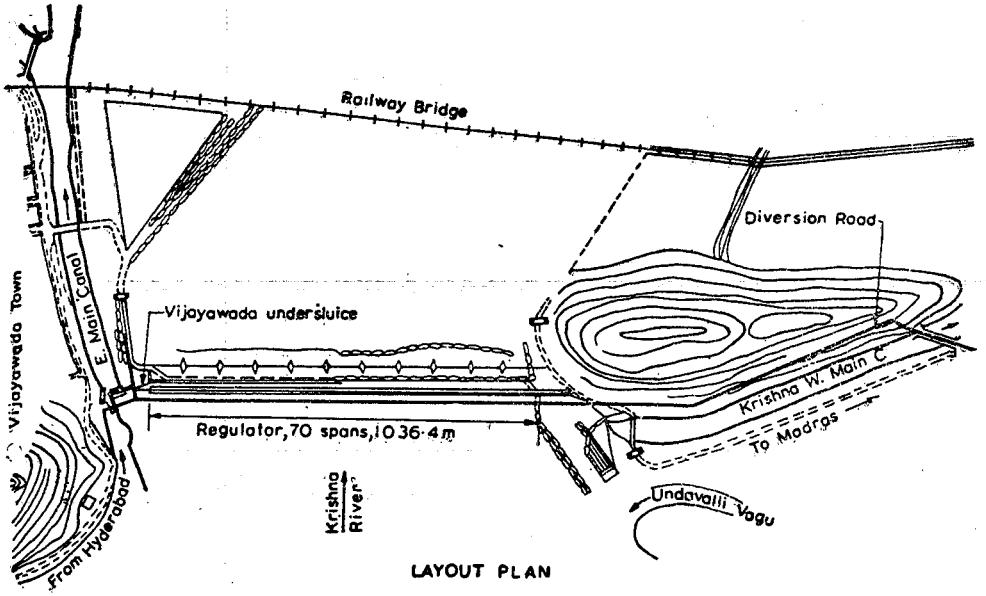


Fig. 11.40. (*b*) Layout plan of Prakasm Barrage.

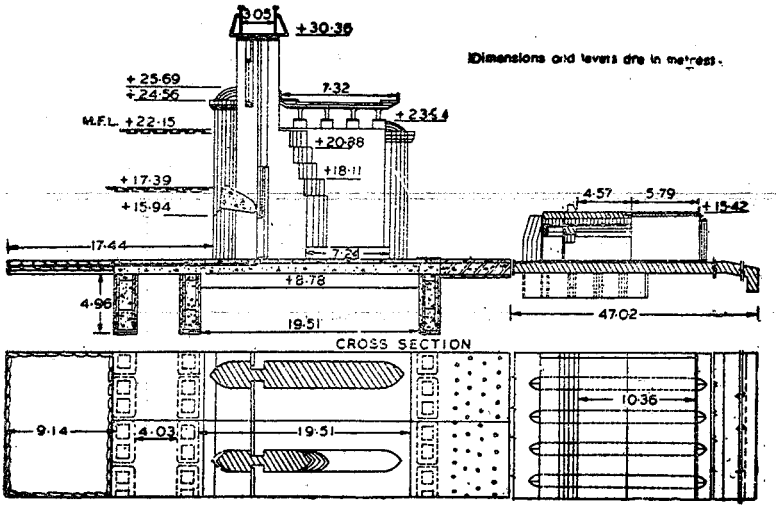


Fig. 11.40. (*c*) Plan (Sitanagram side) of Prakasm Barrage.

Hydraulic Particulars of Prakasm Barrage

Pond Level	...17.38 m
Length of barrage	...1138.73 m
No. of Under-sluice bays	...14
No. of Barrage bays	...70
Width of Under-sluice bays	...5.18 m each
Width of Barrage bays	...12.19 m each
Thickness of intermediate piers	...2.44 m each

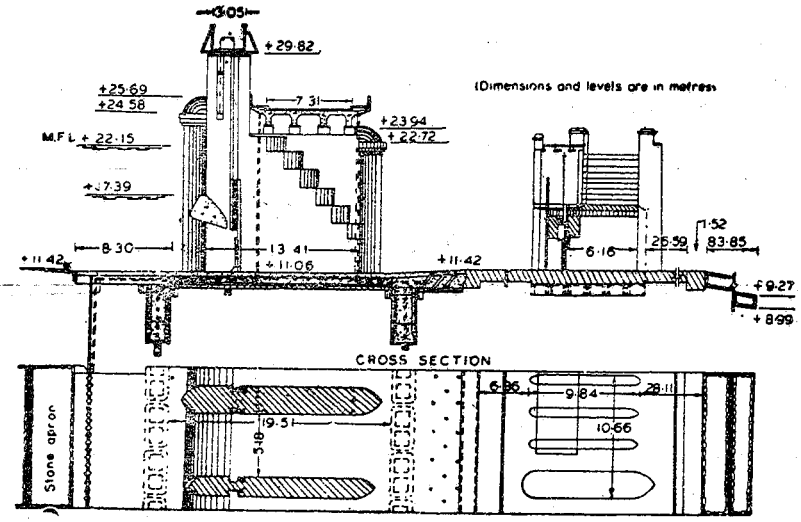


Fig. 11.40. (*d*) Plan (Vijaywada side) of Prakasm Barrage.

Gates

(*a*) *Number*	70
(*b*) *Size*	12.19 m × 3.66 m
Energy Dissipation Arrangements	Cistern

11.7.6. Shah Nehar Barrage. Shar Nehar barrage (Fig. 11.41) is located 5 km downstream of *Beas dam,* in Talwara township (Punjab State) on the Beas river. It caters to the *irrigation* needs of the adjoining areas. The design flood discharge at the barrage site is 11,043 m^3/sec (3.9 lakh cusecs).

Other particulars of this diversion headworks are given below :

Hydraulic Particulars of the Shah Nehar Barrage

Pond level	...330.7 m
Width of the river	...609.6 m
Length of barrage	...561.87 m
No. of Under-sluice bays	...4
No. of Barrage bays	...46
Width of Under-sluice bays	...9.15 m each
Width of Barrage bays	...9.15 m each
Thickness of intermediate pier	...2.13 m (at top)
Gates : (*a*) *Number*	...50
(*b*) *Height*	...6.7 m
Length of Upstream Divide Wall	...41.775 m
Energy Dissipation Arrangements	...Hydraulic jump on cistern

Canal Head Regulator Details

Width of Head regulator	...67.1 m
No. of bays and their width	...8 bays of 7.32 m clear span
Thickness of piers	...1.22 m (at top)
Orientation with respect to barrage axis	at 100°

Energy Dissipation Devices below regulator ...Hydraulic jump in cistern

Max. Discharge of Canal ...325.64 cumecs

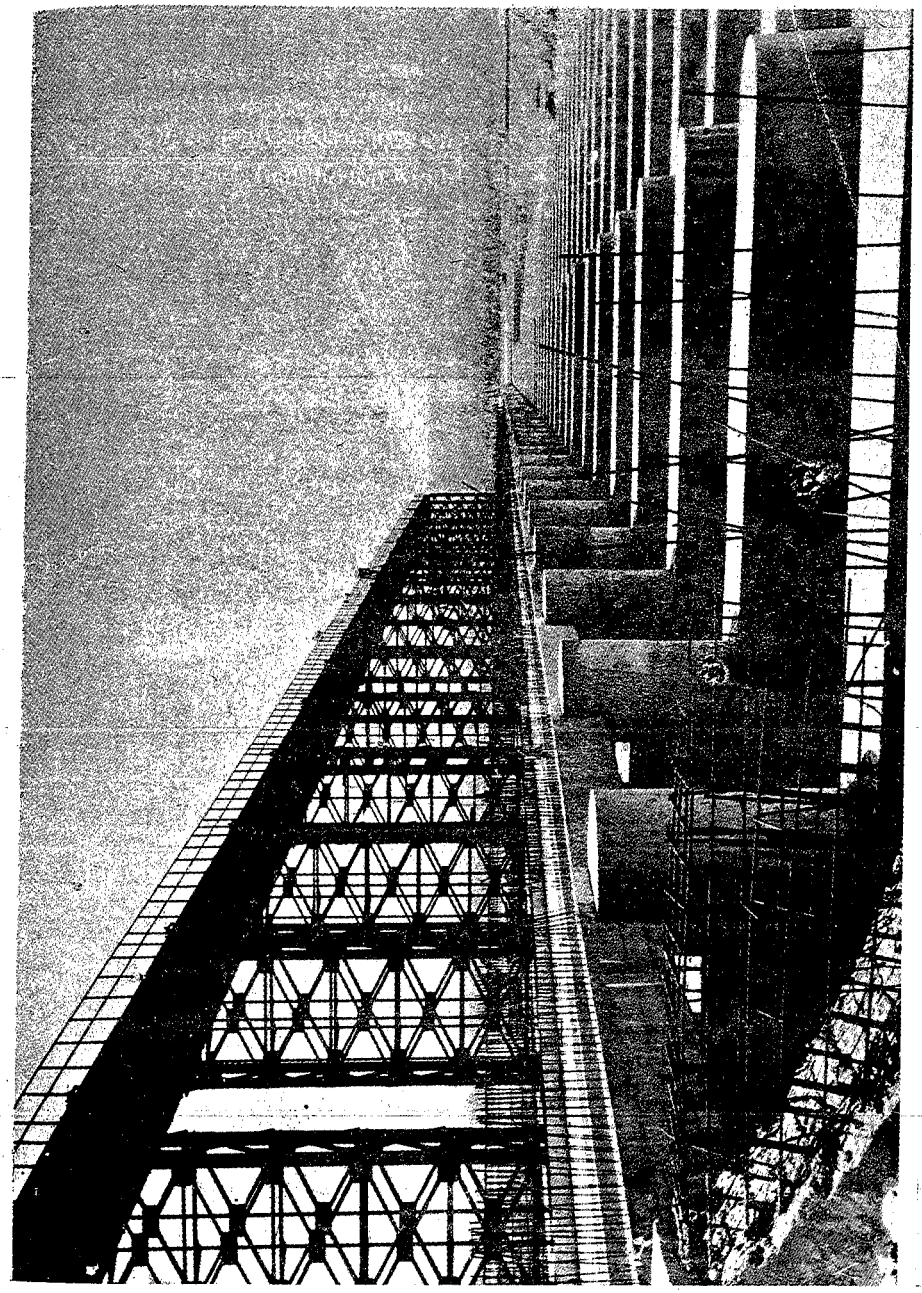


Fig. 11.41. *(a)* Photoview of Shah Nehar Barrage.

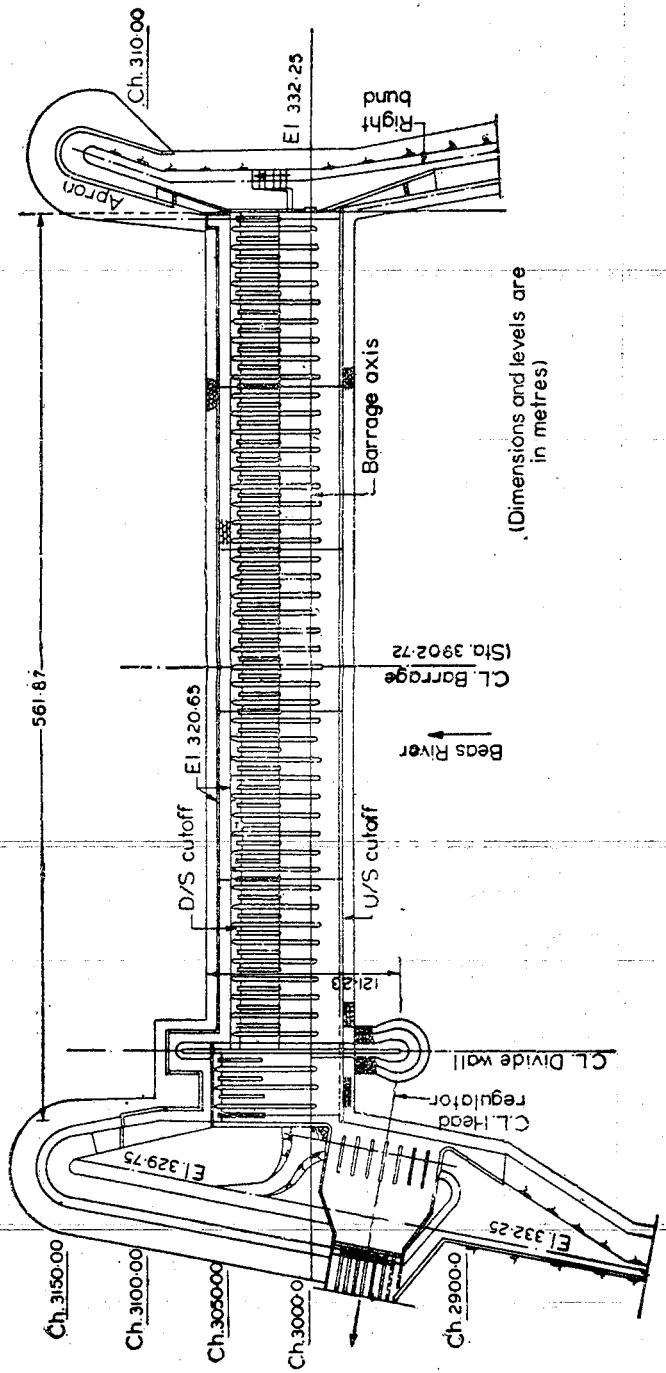


Fig. 11.41 (*b*) Layout plan of Shah Nehar Barrage on *Beas* river in Punjab.

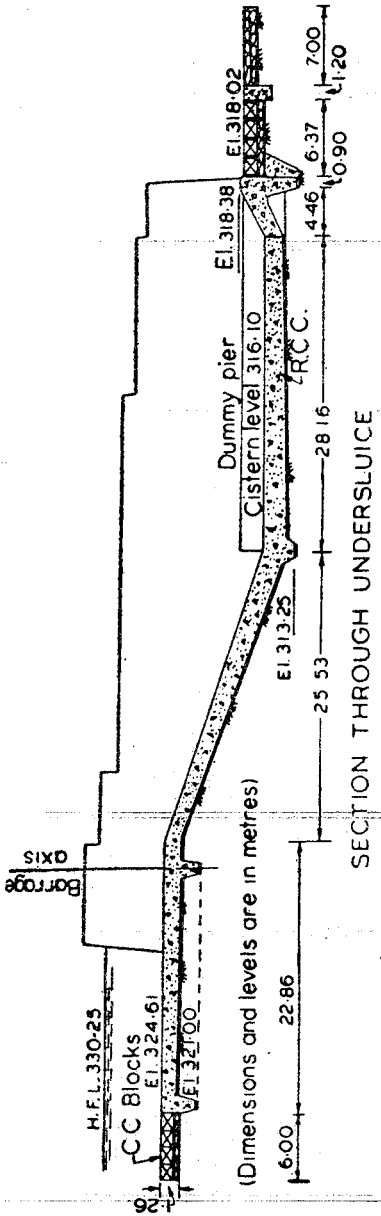


Fig. 11.41. (*c*) Section through Under-sluice bays of Shah Nehar Barrage

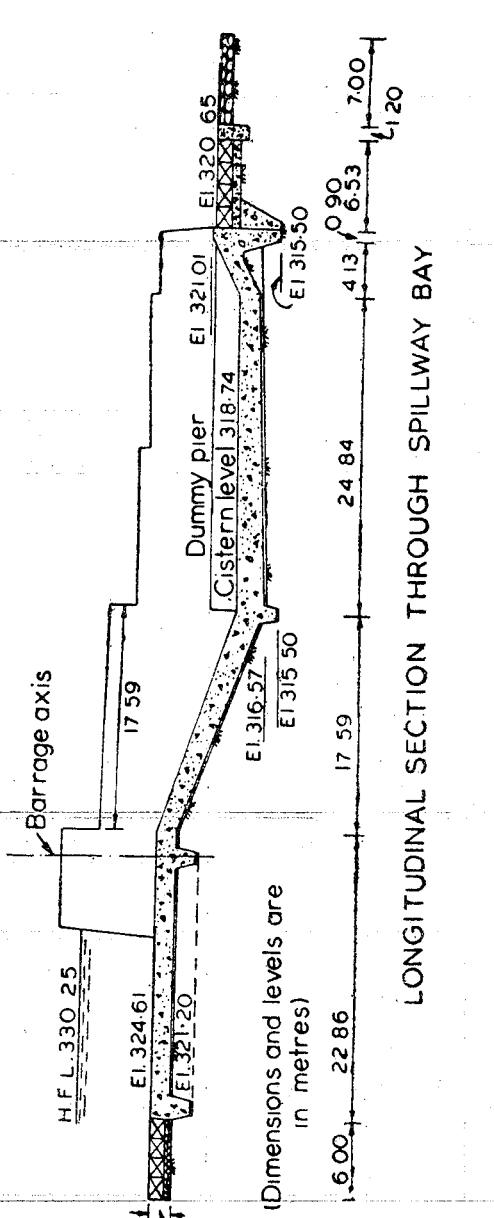


Fig. 11.41. (*d*) Section through Spillway bays of Shah Nehar Barrage.

11.7.7. Tajewala Head Works. Tajewala headworks (Fig. 11.42) was constructed as long back as the year 1873, across Yamuna river. This barrage/anicut is located 37 km from Jagadhri in Haryana, and the famous Western Yamuna Canal (W.J.C.) and the Eastern Yamuna Canal (E.J.C.) take off from this headworks. The design capacity of this barrage is 10,024 cumecs (3.5 lakh cusecs).

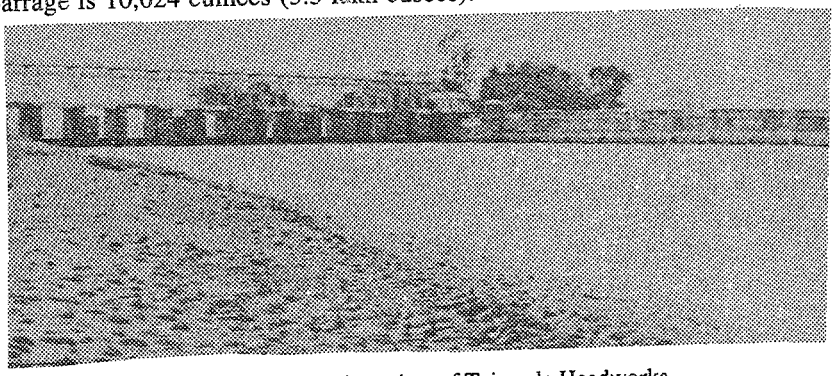


Fig. 11.42. (*a*) Photoview of Tajewala Headworks.

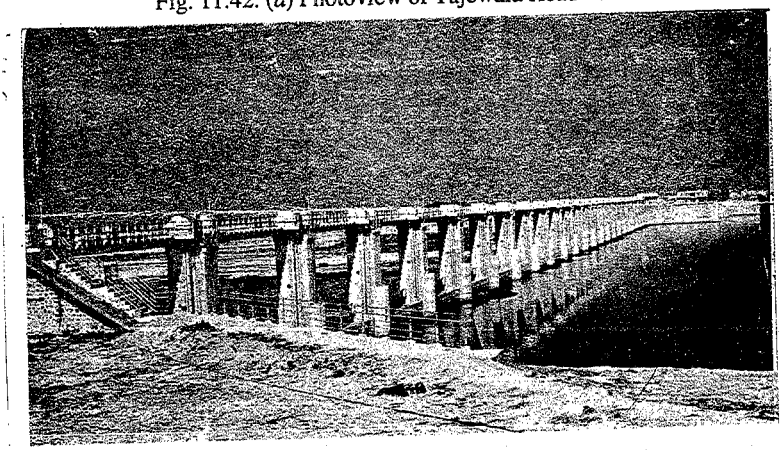


Fig. 11.42. (*b*) Closeview of Tajewala weir bays portion.

Other particulars of this barrage are given below :

Hydraulic Particulars of Tajewala Head works

Length of the barrage/anicut	...753 m
Width of the under sluice bays	...7 spans — 6 m 10 spans—8 m 8 spans—7 m
Width of barrage bays	...564 m
Pond Level	...324 m
Sediment Excluding Devices	...Shingle Excluder
Length of upstream Divide wall	...No. Divide wall
Energy Dissipation devices	...Friction Blocks

Western Yamuna Canal Head Regulator Details

Width of head regulator	...150 m
No. of bays and their width	...13.7 m each and one of 5.4 m
Thickness of piers	...1.37 m

Orientation w.r. to Barrage axis ...90°
Energy Dissipation devices ...Friction Blocks
Max. Discharge of canal ...453 cumecs.

11.7.8. Pick-Up Weir at Ottakkal. This pick-up weir (Fig. 11.43) is located at Ottakkal, Quilon district in Kerala State on Kallada river. The design flood discharge is 2830 cumecs (1 lakh cusecs).

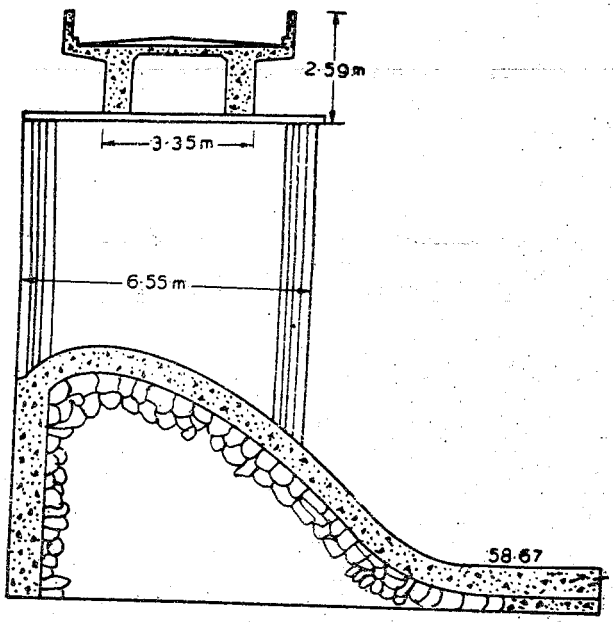


Fig. 11.43. (*a*) Sectional elevation of Ottakkal weir.

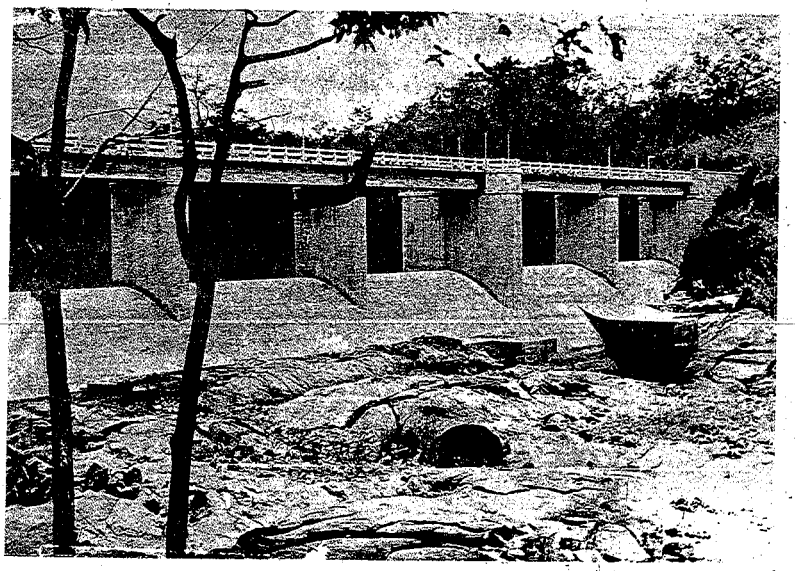


Fig. 11.43. (*b*) Photoview of Ottakkal weir.

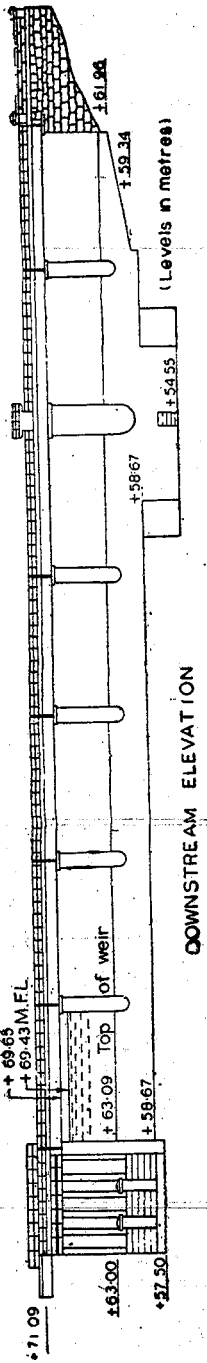


Fig. 11.43. (*c*) Downstream elevation of Ottakal weir.

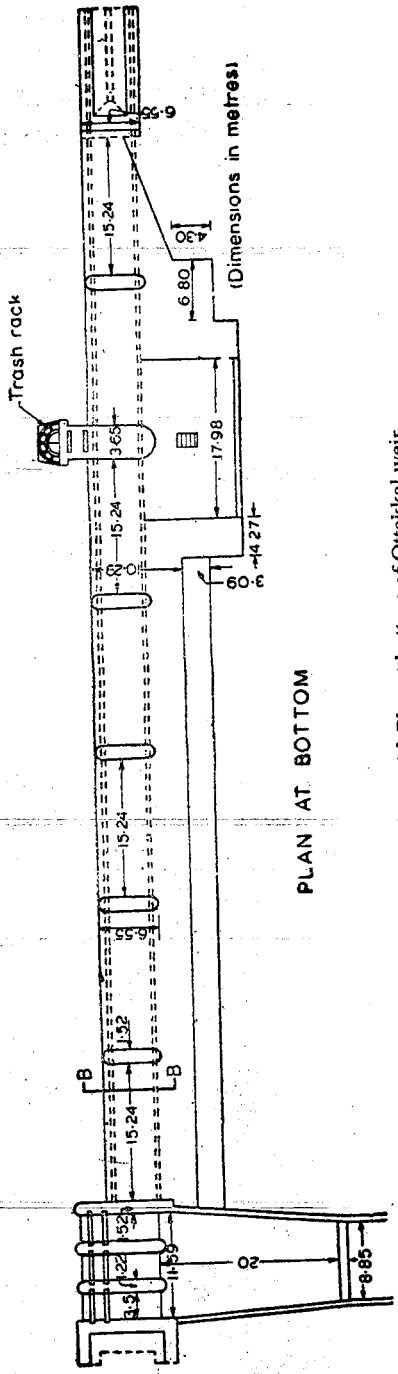


Fig. 11.43. (*d*) Plan at bottom of Ottakkal weir.

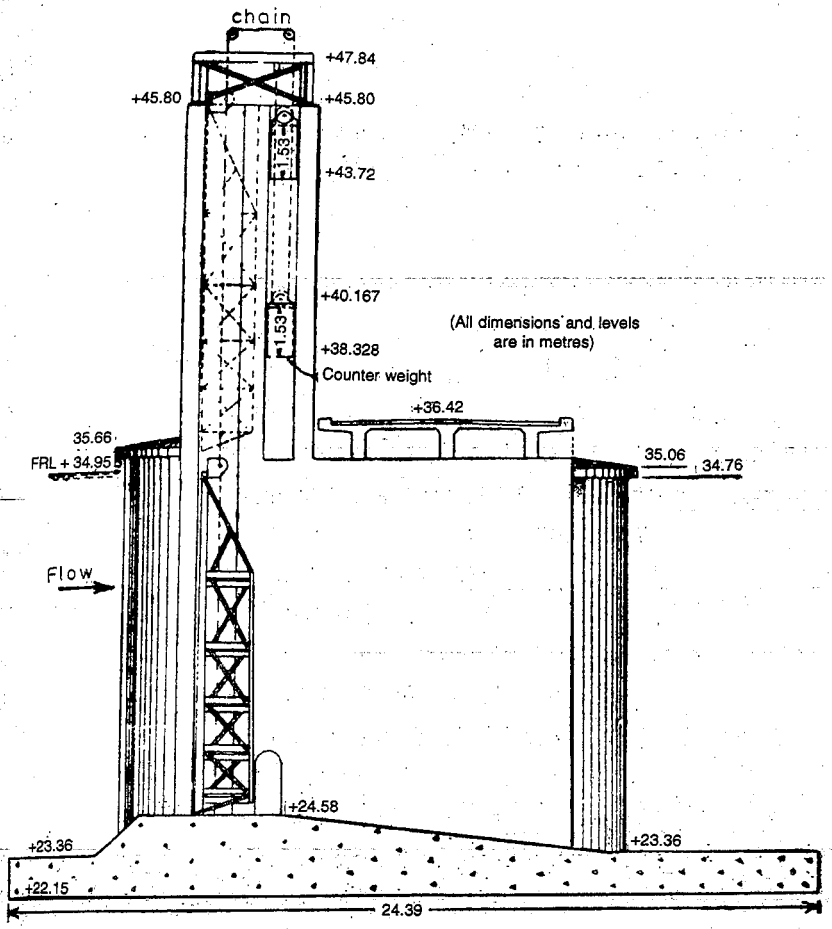


Fig. 11.43. (*e*) Cross-section of Spillway Gate at Ottakkal weir.

Other particulars of this diversion headworks are given below :

Hydraulic Particulars of the Ottakkal weir

Width of the river	...133 m
Length of the weir	...131.04 m
No. of undersluice bays	...1
No. of weir bays	...7, and R.B.C. head regulator bay
Width of undersluice bays	...1.83 m each
Width of weir bays	...15.24 m each
Energy Dissipation Arrangement	...Nil

Canal Head Regulator Details

Width of Head regulator	...11.59 m
No. of bays and their width	...3 of 3.05 m each
Thickness of piers	...1.22 m each
Orientation with respect to weir axis	...90°

Location of Head regulator from weir axis	...R.B.C.* head regulator at the right, and L.B.C.** head regulator at 120 m upstream of the weir axis and 90 m from left edge of the river.
Max. Canal Discharge	...39.08 cumecs (R.B. Canal) 220 cumecs (L.B. Canal)

PROBLEMS

1. (*a*) What are the Main causes of failures of weirs on permeable foundations, and what remedies would you suggest to prevent them ?

(*b*) Explain the surface flow considerations involved in the design of thickness of the sloping glacis and the downstream floor of a weir for different flow considerations.

2. Discuss the main causes of failure of weirs founded on pervious foundation. Also discuss the important theories which have been put forward for designing such weirs to avoid there failure due to the above causes.

3. What are the different types of weirs ? Explain with neat sketches circumstances under which each type is adopted.

What is meant by "piping" in a hydraulic structure ? What are ill-effects of piping ? What are the precautionary methods to avoid the ill effect of piping ? (*Madras University, 1976*)

4. Briefly explain the salient features of Khosla's theory and how it is used in the design of permeable foundations ?

How does Lane's theory differ from Bligh's creep theory ? (*Madras University, 1974*)

5. (*a*) What is meant by "piping" on foundation of a weir. Explain Bligh's method of safe guarding the foundation against the ill effects of piping. (*Madras University, 1975*)

(*b*) Briefly outline Khosla's theory on the design of weirs on permeable foundation. Enumerate the various corrections that are needed in the application of this theory.

6. Explain briefly Khosla's exit gradient concept.

When a weir is constructed across a river ; it disturbs the existing regime. Explain the changes that occur till the river attains regime conditions again. (*Madras University, 1975*)

7. How does Khosla's theory differ from Bligh's theory with regard to the design of weirs on permeable foundations ?

Explain the criteria adopted in designing the various components of a weir built on permeable foundations using Khosla's theory. (*Madras University, 1976*)

8. (*a*) Discuss briefly the causes of failure of hydraulic structures, founded on pervious foundations.

(*b*) State the fundamental difference between Khosla's theory and Bligh's creep theory for seepage below a weir.

(*c*) Following corrected pressure potentials were determined underneath a barrage floor by Khosla's theory :

At junction of upstream sheet pile with floor $\phi_{E_1} = 82\%$

At junction of downstream sheet pile with floor $\phi_{C_2} = 35\%$

Calculate the minimum thickness of the cistern floor at the beginning (*i.e.* at the toe of the glacis), and the end of the cistern (*i.e.* at the junction of the downstream sheet pile with cistern)

The following data are given :

Full reservoir level	= 105 m.
River bed level	= 100 m.
Cistern floor level	= 99 m.

---

* Right Bank Canal

** Left Bank Canal

Total length of barrage between upstream and downstream sheet piles (*i.e.* between E_1 and C_2) = 40 m; Length of cistern = 15 m (from downstream sheet pile).

Assume tail water depth to be nil on the downstream side. Specific gravity of concrete floor = 2.4.

9. (*a*) Draw a neat sectional view of a weir showing the various parts. What is exit gradient ? How does it affect the design of a weir.

(*b*) Following data refer to a weir :

Total number of vertical gates	= 51
Span of each gate	= 10 m.
Full reservoir level (u/s)	= 110 m.
Crest level	= 106 m.
Coefficient of end contraction for piers	= 0.02
Coefficient of discharge (in Francis formula) C_d	= 1.70 $m^{1/2}$/sec.

Compute the max. flood discharge which can safely pass over the weir without exceeding the full reservoir level. Neglect Velocity of approach.

[**Solution** Use $Q = 170 L_e \cdot H^{3/2}$, where $H = 110 - 106 = 4$ m

$L_e = L - [0.1 \times$ No. of end contractions for abutments $+ 0.02 \times$ No. of end contractions for piers$] H$

$L_e = 51 \times 10 - [0.1 \times 2 + 0.02 \times 50] H = 501.2$ m

($\because$ each pier gives 2 contractions, and No. of piers = 50)

$\therefore$ $Q = 1.7 \times 501.2 \times (4)^{3/2}$ cumecs. = **6820 cumecs. Ans.**]

10. Use Khosla's curves to calculate the percentage uplift pressures at the three cut-offs for a barrage foundation profile, shown in Fig. 11.44 applying corrections as applicable.

(Given slope correction for 1 in 4 slope is 3.3%)

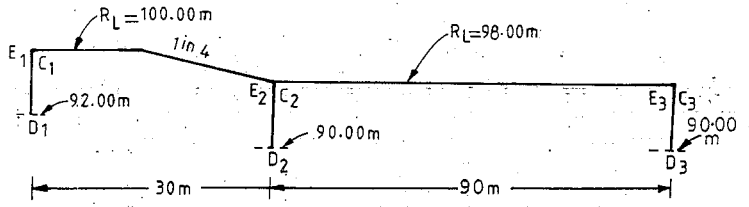


Fig. 11.44

Having determined the percentage uplift pressures, explain how the foundation floor thicknesses are determined corresponding to a known specific gravity of the material of the floor.

11. Write short notes on any three of the following :

(*i*) Stream-lines and Equipotential lines.

(*ii*) Bligh's creep theory for seepage flow.

(*iii*) Khosla's theory and concept of flow net.

(*iv*) Exit gradient and its importance.

(*v*) Khosla's method of independent variables for determining pressures and exit gradient for seepage below a weir.

(*vi*) Design of inverted filters and launching aprons for weirs (u/s as well as d/s).

(*vii*) Retrogression of levels due to weir construction.

(*viii*) Factors governing the design of weirs.

(*ix*) Uplift pressures in jump trough and design of weir floor thickness.

12. Design a barrage across a river at a site which is situated at 25 km downstream of a 120 m high dam, from the following data :

Total length of barrage between upstream and downstream sheet piles (*i.e.* between E_1 and C_2) = 40 m; Length of cistern = 15 m (from downstream sheet pile).

Assume tail water depth to be nil on the downstream side. Specific gravity of concrete floor = 2.4.

9. (*a*) Draw a neat sectional view of a weir showing the various parts. What is exit gradient ? How does it affect the design of a weir.

(*b*) Following data refer to a weir :

Total number of vertical gates	= 51
Span of each gate	= 10 m.
Full reservoir level (u/s)	= 110 m.
Crest level	= 106 m.
Coefficient of end contraction for piers	= 0.02
Coefficient of discharge (in Francis formula) C_d	= 1.70 $m^{1/2}$/sec.

Compute the max. flood discharge which can safely pass over the weir without exceeding the full reservoir level. Neglect Velocity of approach.

[**Solution** Use $Q = 170\, L_e \cdot H^{3/2}$, where $H = 110 - 106 = 4$ m

$L_e = L - [0.1 \times$ No. of end contractions for abutments $+ 0.02 \times$ No. of end contractions for piers$]\, H$

$L_e = 51 \times 10 - [0.1 \times 2 + 0.02 \times 50]\, H = 501.2$ m

($\because$ each pier gives 2 contractions, and No. of piers = 50)

$\therefore$ $Q = 1.7 \times 501.2 \times (4)^{3/2}$ cumecs. = **6820 cumecs. Ans.**]

10. Use Khosla's curves to calculate the percentage uplift pressures at the three cut-offs for a barrage foundation profile, shown in Fig. 11.44 applying corrections as applicable.

(Given slope correction for 1 in 4 slope is 3.3%)

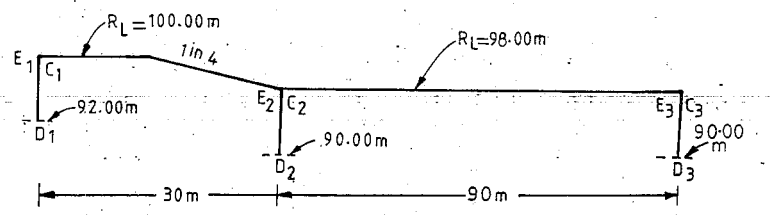


Fig. 11.44

Having determined the percentage uplift pressures, explain how the foundation floor thicknesses are determined corresponding to a known specific gravity of the material of the floor.

11. Write short notes on any three of the following :

(*i*) Stream-lines and Equipotential lines.

(*ii*) Bligh's creep theory for seepage flow.

(*iii*) Khosla's theory and concept of flow net.

(*iv*) Exit gradient and its importance.

(*v*) Khosla's method of independent variables for determining pressures and exit gradient for seepage below a weir.

(*vi*) Design of inverted filters and launching aprons for weirs (u/s as well as d/s).

(*vii*) Retrogression of levels due to weir construction.

(*viii*) Factors governing the design of weirs.

(*ix*) Uplift pressures in jump trough and design of weir floor thickness.

12. Design a barrage across a river at a site which is situated at 25 km downstream of a 120 m high dam, from the following data :

High flood discharge = 6100 cumecs.
High flood level (before the construction of barrage) = 229.9 m.
River bed level (winter) = 224.3 m.
Pond level = 228.15 m.
Permissible afflux = 1.0 m.
Canal discharge = 151.5 cumecs.
Canal bed width = 50 m
Canal full supply depth = 3 m.
Canal bed level = 224 m.
Angle of off-take = 110°
Silt factor in the river = 1.5

The gauge discharge curve of the river is given below in Fig. 11.45.

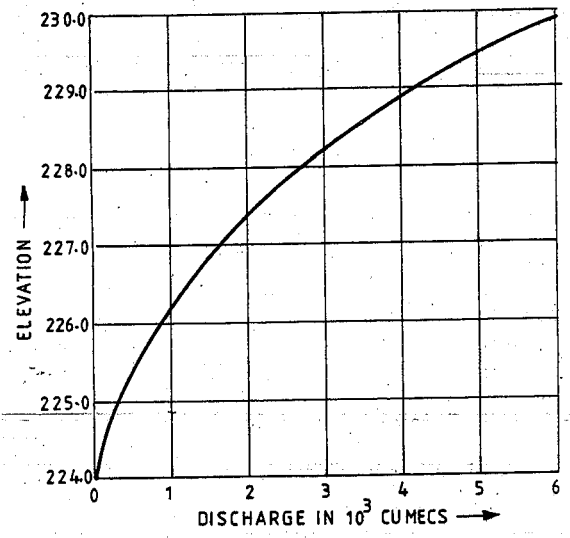


Fig. 11.45

12

Canal Falls

12.1. Definition and Location of Canal Falls

12.1.1. Definition. Whenever the available natural ground slope is steeper than the designed bed slope of the channel, the difference is adjusted by constructing vertical 'falls' or 'drops' in the canal bed at suitable intervals, as shown in Fig. 12.1.

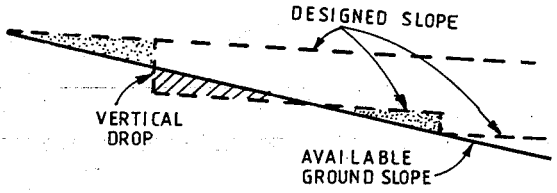


Fig. 12.1

Such a drop in a natural canal bed will not be stable and, therefore, in order to retain this drop, a masonry structure is constructed. Such a pucca structure is called a *canal fall* or a *canal drop.*

12.1.2. Proper location. The location of a fall in a canal depends upon the topography of the country through which the canal is passing. In case of the main canal, which does not directly irrigate any area, the site of a fall is determined by considerations of economy in 'cost of excavation and filling' *versus* 'cost of fall'. The excavation and filling on two sides of a fall should be tried to be balanced, because the unbalanced earthwork is quite costly. By providing a larger drop in one step, the quantity of unbalanced earth work increases, but at the same time, the number of fall reduces. An economy between these two factors has to be worked out before deciding the locations and extent of falls.

In case of branch canals and distributary channels, the falls are located with consideration to commanded area. The procedure is to fix the FSL required at the head of the off-taking channels and outlets and mark them on the L-section of the canal. The FSL of the canal can then be marked, as to cover all the commanded points, thereby deciding suitable locations for falls in canal FSL, and hence, in canal beds.

The location of the falls may also be influenced by the possibility of combining it with a bridge, regulator, or some other masonry work, since such combinations often result in economy and better regulation. When a fall is combined with a regulator and a bridge, it is called a fall-regulator with road bridge.

12.2. Types of Falls

Various types of falls have been designed and tried since the inception of the idea of 'falls construction' came into being. The important types of such falls, which were used in olden days and those which are being used in modern days, are described below:

(1) Ogee Falls. The 'Ogee type fall' was constructed in olden days on projects like Ganga canal. The water was gradually led down by providing convex and concave curves, as shown in Fig. 12.2.

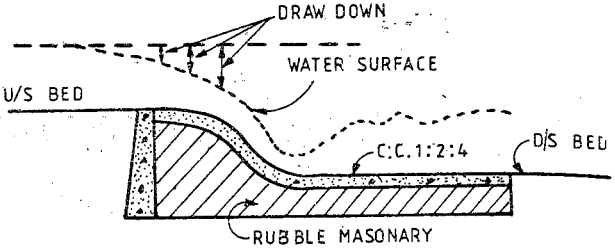


Fig. 12.2. Ogee fall.

The performance of such a fall was found to have the following major defects :

(*i*) There was heavy draw-down on the upstream side, resulting in lower depths, higher velocities and consequent bed erosion. Draw-down would also affect the supply in a distributary, situated just upstream of fall.

(*ii*) Due to smooth transition, the kinetic energy of the flow was not at all dissipated, causing erosion of downstream bed and banks.

A 'raised crest' was soon added to restrict the draw-down and a long protection was provided on the downstream side. Later, it was converted into a much better type of fall, called a 'Vertical Impact type'.

(2) Rapids. In Western Yamuna canal, long rapids at slopes of 1 : 15 to 1 : 20 (*i.e.*, gently sloping glacis) with boulder facings, were provided. They worked quite satisfactorily, but were very expensive, and hence became obsolete.

(3) Trapezoidal Notch Falls. The trapezoidal notch fall was designed by Ried in 1894. It consists of a number of trapezoidal notches constructed in a high crested wall across the channel with a smooth entrance and a flat circular lip projecting downstream from each notch to spread out the falling jet (See Fig. 12.3).

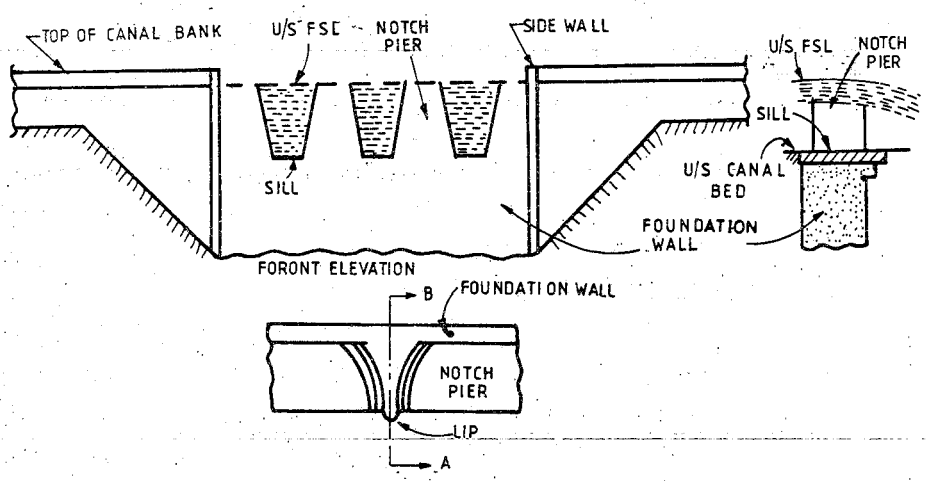


Fig. 12.3. Trapezoidal Notch fall.

The notches could be designed to maintain the normal water depth in the upstream channel at any two discharges, as the variation at intermediate values is small. Hence, the depth discharge relationship of the channel remains practically unaffected by the introduction of the fall. In other words, there would neither be drawdown nor heading

up of water, as the channel approaches the fall. These falls remained quite popular, till simpler, economical, and better modern falls were developed.

(4) **Well Type Falls or Cylinder Falls or Syphon Well Drops.** This type of a fall consists of an inlet well with a pipe at its bottom, carrying water from the inlet well to a downstream well or a cistern. The downstream well (as shown in Fig. 12.4) is necessary in the case of falls greater than 1.8 m and for discharges greater than 0.29 cumecs. The water falls into the inlet well, through a trapezoidal notch constructed in the steining of the well, from where it emerges near the bottom, dissipating its energy in turbulence inside the well.

This type of falls are very useful for affecting larger drops for smaller discharges. They are commonly used as tail escapes for small canals, or where high levelled smaller drains do outfall into a low levelled bigger drain.

(5) **Simple Vertical Drop Type and Sarda Type Falls.** A raised crest fall with a vertical impact (Fig. 12.5) was first of all introduced on Sarda Canal System in U.P., owing to its economy and simplicity. The necessity for economic falls arose because of the need of construction of a large number of smaller falls on the Sarda Canal System. In that area, a thin layer of sandy clay overlied a stratum of pure sand. If the canal bed was to be cut deep and up to the sand stratum, the seepage losses would have been tremendous. Hence, the depth of cutting had to be kept low, necessitating the construction of a large number of smaller falls.

In this type of a high crested fall, the nappe impinges into the water cushion below. *There is*

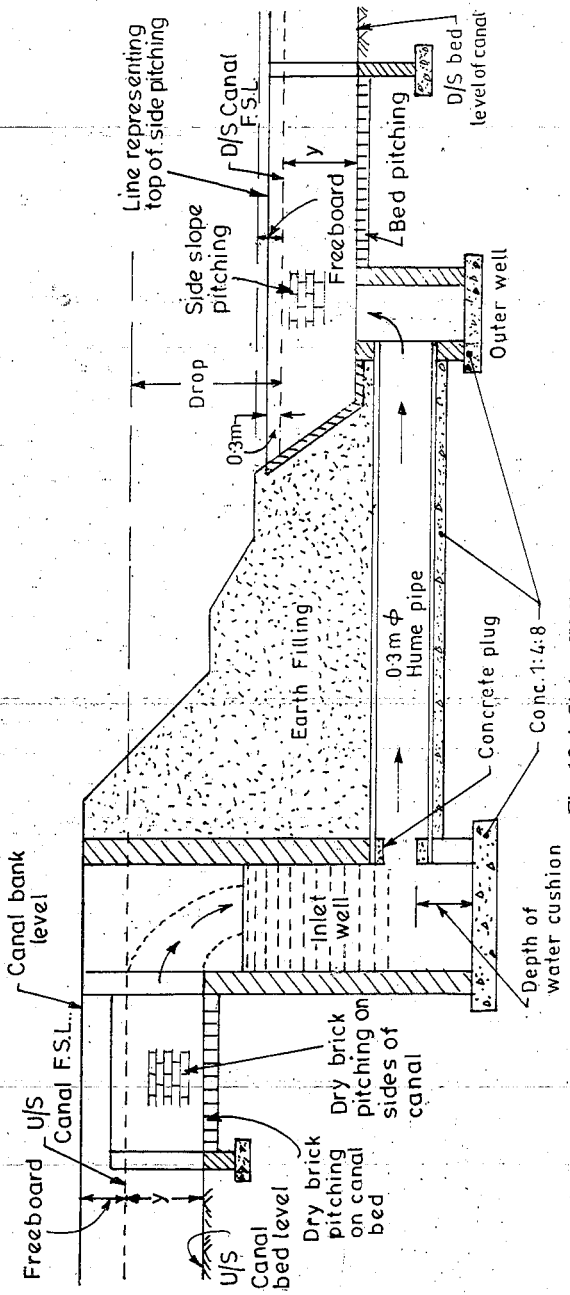


Fig. 12.4. Siphon Well drop.

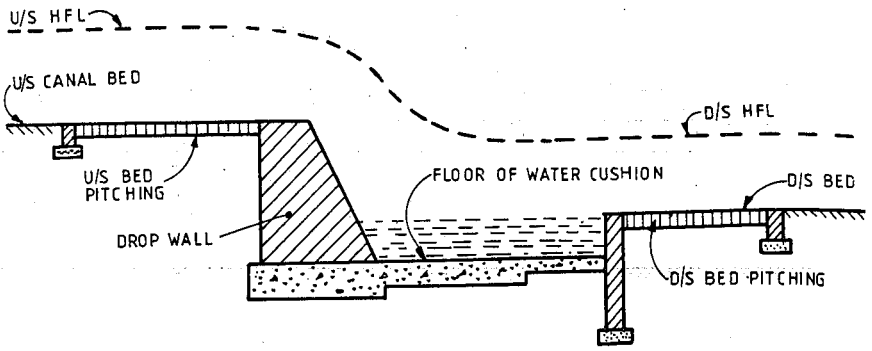


Fig. 12.5. Simple Vertical Drop fall.

no clear hydraulic jump and the energy dissipation is brought about by the turbulent diffusion, as the high velocity jet enters the deep pool of water downstream.

Two types of crests which are used in Sarda type falls are shown a little later in Figs. 12.10 (*a*) and (*b*).

Sarda type fall is a high crested fall, and if the discharge in the canal varies (say between 50 to 100%), the water will head up on the upstream side at low discharges. The reach upstream of the crest will silt up as the clearer water will pass downstream of the crest. Due to reduction in silt in the d/s discharge, there may be a tendency of scouring on d/s, so as to make up the silt loss. Hence, this type of fall is not quite suitable for canals in which discharge varies within a wide range. A trapezoidal notch fall, although costlier than Sarda type or glacis type fall, is free from such troubles and, therefore, preferred for canals where the discharge is very small and also varies over a wide range.

(6) Straight Glacis Falls. In this type of a modern fall, a 'straight glacis' (generally sloping 2 : 1) is provided after a 'raised crest' (see Fig. 12.6). The hydraulic jump is

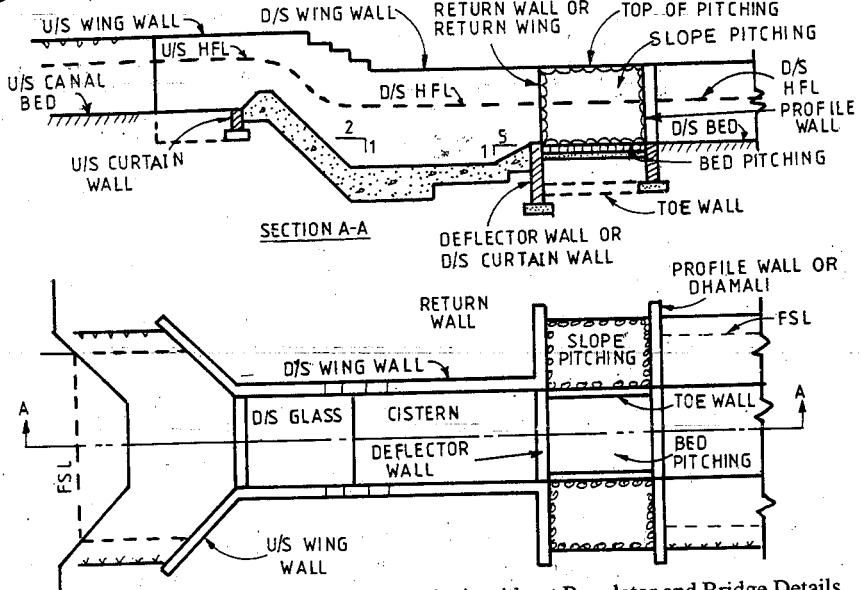


Fig. 12.6. *'Straight Glacis fall'* (without fluming), without Regulator and Bridge Details.

made to occur on the glacis, causing sufficient energy dissipation. This type of falls give very good performance if not flumed, although they may be flumed for economy. They are suitable up to 60 cumecs discharge and 1.5 m drop.

(7) **Montague Type Falls.** The energy dissipation on a straight glacis remain incomplete due to vertical component of velocity remaining unaffected. An improvement in energy dissipation may be brought about in this type of fall [see Fig. 12.7 (*a*)], by replacing the straight glacis by a parabolic glacis, commonly known as 'Montague Profile'.

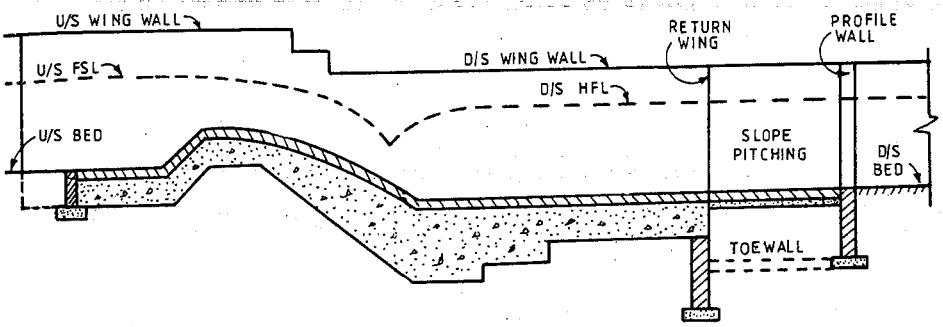


Fig. 12.7. (*a*) Montague Type fall.

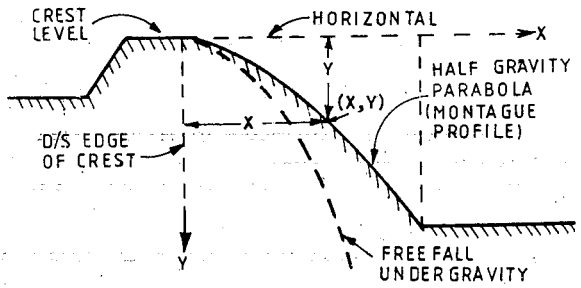


Fig. 12.7. (*b*) Montague Profile.

The Montague profile is given by the equation.

$$X = U\sqrt{\frac{4Y}{g}} + Y \qquad ...(12.1)$$

where X = The horizontal ordinate of any point of the profile measured from the d/s edge of crest.

Y = Vertical ordinate measured from the crest level.

U = Initial velocity of water leaving the crest.

The curved glacis is difficult to construct and thereby rendering it costlier. This type of fall is, therefore, generally not adopted in India.

(8) Inglis Falls or Baffle Falls. A straight glacis type fall when added with a baffle platform and a baffle wall as shown in Fig. 12.8, was developed by Englis, and is called 'Englis Fall' or 'Baffle Fall'. They are quite suitable for all discharges and for drops of more than 1.5 m. They can be flumed easily as to affect economy.

The baffle wall is provided at a calculated height and a calculated distance from the toe of the glacis, so as to ensure the formation of the jump on the baffle platform, as shown in Fig. 12.8.

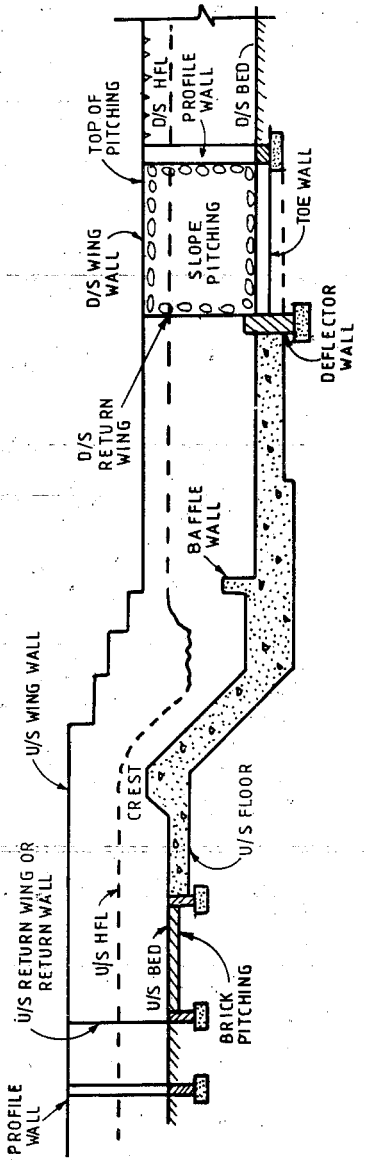


Fig. 12.8. 'Baffle fall' or 'English-fall'.

Comparison of Different Types of Falls. Extensive testing of various modern types of falls was carried out in 1952 at Research Station Poona, and the following recommendations were made :

(*i*) 'Vertical drop' falls are quite suitable *for discharges upto 15 cumecs and drops upto. 1.5 metres.* But this type of fall should not be flumed.

(*ii*) For 'Straight glacis' type falls, the conclusion was, that they work satisfactorily for all conditions, if unflumed ; but in that case they become costly. *Even then, they can be adopted suitably for discharges upto 60 cumecs and drops upto 1.5 metres, and can even be flumed.*

(*iii*) 'Baffle fall' or 'English fall' may be used for all discharges when drop is more than 1.5 metres. *This type of fall functions very satisfactorily, either flumed or unflumed, so long as it is undrowned.*

(*iv*) Well type falls are suitable and economical for high drops and very low discharges. They can hence be easily used, as tail escapes of small channels.

Meter and Non-Meter Falls. Meter falls are those which can be used to measure the discharge of the canal. If the discharge cannot be measured accurately at the site of the fall, then it is called a non-meter fall. Vertical drop fall is not suitable as a meter due to the formation of partial vacuum under the nappe. Glacis type fall is quite suitable as a metering device. *Since a sharp crest does not give a constant coefficient of discharge with varying heads, while a broad crest does so reasonably ; a fall to be used as meter must be provided with a broad crest.* Generally, a flumed glacis fall or a flumed baffle fall, is used as meter, while an unflumed glacis fall is used as a non-meter fall.

DESIGN PRINCIPLES OF VARIOUS TYPES OF FALLS

12.3. Design of a Trapezoidal Notch Fall

As pointed out earlier, a notch fall provides a proportionate fall, in the sense that there is no heading up or drawdown of water level in the canal near the fall. The whole width of the channel is divided into a number of notches. The crest (*i.e.* the sill level or the level of the bottom of notch) may be kept higher than the bed level of the canal, which will tend to increase the length of the weir, but in no case, the total length of the weir openings should exceed the bed width of the canal upstream, and may well be reduced to about $\frac{7}{8}$th of the bed width.

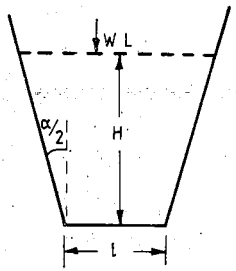


Fig. 12.9
Trapezoidal notch

Discharge Formula. The discharge passing through one notch of a notch fall can be obtained by adding the discharge of a rectangular notch and a V-notch.

∴ The discharge passing through a trapezoidal notch such as shown in Fig. 12.9 is given by

$$Q = \frac{2}{3} C_d \cdot \sqrt{2g}\ l\ .\ H^{3/2} + \frac{8}{15} \cdot C_d \cdot \sqrt{2g} \tan \frac{\alpha}{2} H^{5/2}$$

$$= \frac{2}{3} C_d \cdot \sqrt{2g} \left[lH^{3/2} + \frac{4}{5} \tan \frac{\alpha}{2} H^{5/2} \right]$$

$$= \frac{2}{3} C_d \cdot \sqrt{2g} \left[lH^{3/2} + \frac{2}{5} \left(2 \tan \frac{\alpha}{2} \right) H^{5/2} \right] \qquad ...(12.2)$$

If $2 \tan \frac{\alpha}{2}$ is represented by n, then

$$Q = \frac{2}{3} C_d \cdot \sqrt{2g} \left[lH^{3/2} + 0.4\ .\ nH^{5/2} \right] \qquad ...(12.3)$$

where, C_d = Coefficient of discharge ≈ 0.75

$$\therefore \quad Q = \frac{2}{3} \times 0.75 \sqrt{2 \times 9.81} \left[lH^{3/2} + 0.4\, nH^{5/2} \right]$$

or $$Q = 2.22 H^{3/2} \left[l + 0.4nH \right] \qquad ...(12.4)$$

The above discharge equation contains two unknowns l and n. For solving this equation, two values of Q and corresponding values of H must be assumed. It is a common practice to design notches for full supply discharge (Q_{100}) and half supply discharge (Q_{50}) with values of H equal to the normal water depths in the channel in the respective cases. Let the normal water depths in the channel at full discharge and half discharge be represented by y_{100} and y_{50} respectively. Then $H_{100} = y_{100}$, and $H_{50} = y_{50}$.

The depth of water in the channel at 50% discharge (*i.e.* y_{50}) can be approximately evaluated in terms of full supply depth (y_{100}) as follows :

Let $V = C \cdot y^{0.64}$ ...(Kennedy's Eq. for Vel. in channels)

Now $Q = A.V.$

$\therefore$ $Q = B\ .\ y\ .\ C\ .\ y^{0.64}$ Using $A \approx B.y$ (neglecting sy^2)

or $Q = C\ .\ B\ .\ y^{1.64}$

$\therefore$ $Q_{100} = C\ .\ B\ .\ y_{100}^{1.64}$

and $$Q_{50} = C \,.\, B \,.\, y_{50}^{1.64}$$

or $$\frac{Q_{50}}{Q_{100}} = \left(\frac{y_{50}}{y_{100}}\right)^{1.64}$$

or $$\frac{y_{50}}{y_{100}} = \left(\frac{Q_{50}}{Q_{100}}\right)^{\frac{1}{1.64}} = (0.5)^{\frac{1}{1.64}} = 0.66$$

$$\therefore \quad y_{50} = 0.66 \cdot y_{100} \qquad ...(12.5)$$

Number of Notches. The number of notches should be so adjusted by hit and trial method that the top width of the notch lies between $\frac{3}{4}$ to full water depth above the cill of the notch. This hit and trial procedure would become clear when we solve a numerical example.

Notch Piers. The thickness of notch piers should not be less than half the water depth and may be kept more if they have to carry a heavy super structure. The top length of piers should not be less than their thickness.

In plan, the notch profile is set back by 0.5 m from the downstream face of the notch fall for larger canals, and by 0.25 m for distributaries. All curves are circular arcs, and all centres lie in the plane of the profile. The splay upstream from the notch section is 45°, and the downstream splay is kept 22.5°. The lip is circular and is corbelled out by 0.8 m on larger canals, and by 0.6 m on distributaries.

Example 12.1. *Design the size and number of notches required for a canal drop with the following particulars :*

Full supply discharge	*= 4 cumecs*
Bed width	*= 6.0 m*
F.S. depth	*= 1.5 m*
Half supply depth	*= 1.0 m*

Assume any other data if required. *(Madras University 1975)*

Solution. The bed width of the canal is 6 m. Each notch at top should be roughly equal to F.S. depth *i.e.* 1.5 m. So let us, in the first trial, provide 3 notches.

$\therefore$ Full supply discharge through each notch $= \frac{4}{3} = 1.33$ cumecs

From Eq. (12.4) we have

$$Q = 2.22 H^{3/2} [l + 0.4\, nH]$$

Using $Q_{100} = 2.22\, (y_{100})^{3/2} [l + 0.4n\, y_{100}]$

where $Q_{100} = 1.33$ cumecs

$y_{100} = 1.5$ m

$\therefore$ We have $1.33 = 2.22 \,.\, (1.5)^{3/2} [l + 0.4n \times 1.5]$

or $1.33 = 2.22 \times 1.84\, [l + 0.6n]$

or $l + 0.6n = 0.326 \qquad ...(i)$

Now, using

$$Q_{50} = 2.22 \,.\, (y_{50})^{3/2} [l + 0.4n \,.\, y_{50}]$$

where $Q_{50} = \frac{1.33}{2} = 0.67$ cumecs

$y_{50} = 1.0$ m

$$\therefore \quad 0.67 = 2.22 \,.\, (1.0)^{3/2} [l + 0.4n \times 1]$$

or $$l + 0.4n = 0.3 \qquad ...(ii)$$

Subtracting (*ii*) from (*i*) we get

$$0.2n = 0.026$$

or $$n = 0.13.$$

Putting the value of n in (*ii*) we get

$$l + 0.4 \times 0.13 = 0.3$$

or $$l = 0.248; \text{ say. } \mathbf{0.25\ m.}$$

By this trial, we find the top width

$$= 0.25 + 2 \tan \alpha \cdot H = 0.25 + n.H$$
$$= 0.25 + 0.13 \times 1.5 = 0.25 + 0.195$$
$$= 0.445; \text{ say } 0.45 \text{ m, which is much less than the full depth of } 1.5 \text{ m.}$$

In order to increase the top width, and to make it near 1 to $\frac{3}{4}$th *FSD*, it is necessary to increase l and n : which can be done by reducing the number of notches.

The values of l and n obtained for 3 notches will increase in direct proportion, when number of notches are reduced.

In other words, the values l of and n will become 3 times, when number of notches are reduced 3 times. Thus, when we provide only one notch instead of 3 notches, the values of n and l will triple. Similarly, when we use 2 notches against 3, *i.e.*, $\frac{1}{1.5}$ times the values n and l will become 1.5 times of those obtained for 3 notches.

Hence when we use 2 notches, values will be

$$n = 1.5 \times 0.13 = 0.20$$
$$l = 1.5 \times 0.25 = 0.3 \text{ m}$$

and Top width $= 1.5 \times 0.45 = \mathbf{0.68\ m.}$

Since the width is still quite low, we may use only one notch.

When we use only one notch, the values will be

$$n = 3 \times 0.13 = 0.39$$
$$l = 3 \times 0.25 = 0.75 \text{ m}$$
$$\text{Top width} = 3 \times 0.45 = 1.35 \approx \text{FSD (O.K.)}$$
$$\frac{\alpha}{2} = \tan^{-1} \frac{n}{2} = \tan^{-1} \frac{0.39}{2} = 11°$$

Since this condition gives us top width = 1.35 m, which is O.K., we may provide one notch, centrally placed in the given channel of 6 m width. The section of the notch to be adopted is also shown in Fig. 12.10.

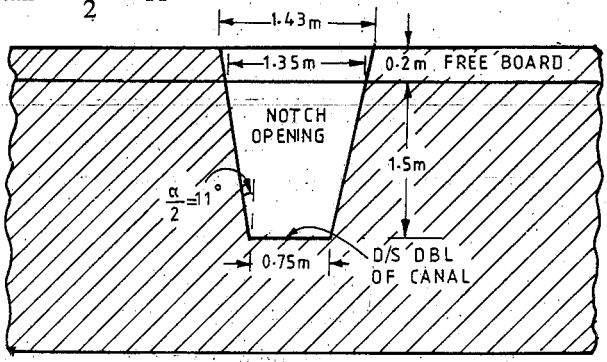


Fig. 12.10

Check for raised crest if possible. It has also been noticed that when lesser

number of notches are provided, with their bottoms kept at U/S DBL of canal, the concentration of flow gets increased considerably. To avoid such an eventuality, it is preferable to increase the number of notches, and this may sometimes be achieved by providing the notches in the raised crest. In other words, the bottom of notch opening will be kept higher than U/S DBL of canal. This raising may be between 10% to 30% of full depth. The design calculations are hence to be repeated to compute n and l with a raised crest, whenever a detailed designing is being done, and number of notches determined are low.

These calculations for the above question 12.1 will be as follows :

Let us assume a raised crest equal to 20% of FSD

$$= 20\% \times 1.5 \text{ m} = 0.3 \text{ m}.$$

Using Eqn. (12.4) as

$$Q = 2.22\, H^{3/2}\,[l + 0.4nH], \text{ we have}$$

$$Q_{100} = 2.22\,(1.5 - 0.3)^{3/2}\,[l + 0.4n\,(1.5 - 0.3)]$$

or $$Q_{100} = 2.92\,(l + 0.48n) \qquad ...(iii)$$

Also $$Q_{50} = 2.22\,(1.0 - 0.3)^{3/2}\,[l + 0.4n\,(1.0 - 0.3)]$$

[$\because$ FSD at $\frac{1}{2}$ discharge = 1.0 m (given)]

or $$Q_{50} = 1.3\,(l + 0.28\,n) \qquad ...(iv)$$

But $$Q_{100} = 2Q_{50}$$

$$\therefore \quad 2.92\,(l + 0.48\,n) = 2 \times 1.3\,(l + 0.28\,n)$$

This gives a negative value of n, which is not feasible, and hence such a raised crest may not be feasible in this particular case. Hence, the design made earlier, and shown in Fig. 12.10, holds good.

12.4. Design of a Syphon Well Drop

A syphon well drop, such as shown in Fig. 12.4, is generally adopted for smaller discharges and larger drops. The main features of the design involves determining the size of the inlet well and that of the hume pipe. A suitable size for the outer well, a proper provision of water cushion at the bottom of the inlet well, the bed and side slope pitchings in the canal upstream as well as downstream for suitable lengths, are also provided. The size of the inlet well and that of the syphon pipe are determined on the following considerations w.r. to Fig. 12.11.

First of all, the size of the trapezoidal notch is determined so as to pass the designed discharge by using eqn. (12.4) in the same way as

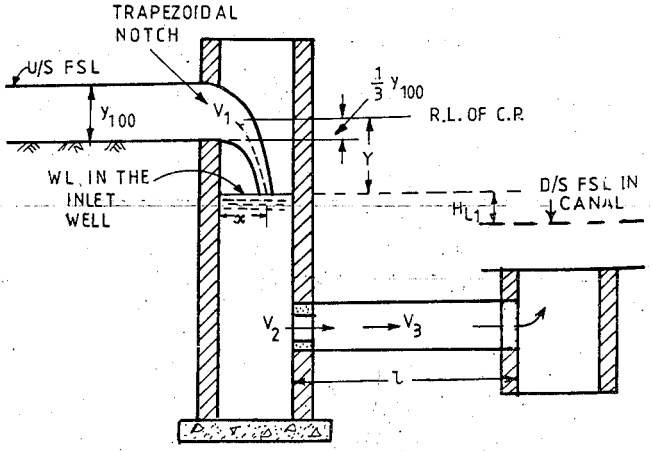


Fig. 12.11

is done for a trapezoidal notch. Then let V_1 be the velocity over the notch, V_2 be the velocity of entry in the pipe, and V_3 be the velocity through the pipe. All these values of velocities can be determined easily as below :

$$V_1 = \frac{\text{Full supply discharge}}{\text{Area of flow over the notch}}$$

$$V_2 = \frac{\text{Full supply discharge}}{\text{Area of opening at entry (for assumed dia of opening)}}$$

$$V_3 = \frac{\text{Full supply discharge}}{\text{Area of pipe (for assumed dia)}}$$

The head loss between the inlet well and the d/s FSL is then given by H_{L_1} as

$$H_{L_1} = 0.5\frac{V_2^2}{2g} \text{ (i.e. loss due to entry)} + \frac{(V_2 - V_3)^2}{2g} \text{ (i.e. the loss due to sudden enlargement)} + \frac{f' L V_3^2}{2gd} \text{ (i.e. the loss in the assumed pipe length } L\text{)} + \frac{V_3^2}{2g} \text{ (loss due to exit).} \quad ...(12.6)$$

Knowing all the above values, H_{L_1} can be determined, and thus the R.L. of water surface inlet well (*i.e.* d/s FSL + H_{L_1}) can be determined .

Now, approximate *R.L.* of centre of pressure (C.P.) of the trapezoidal waterway through the notch

$$= \text{u/s canal bed level} + \frac{1}{3}\text{FSD.}$$

= (which can be calculated)

Then, the height (Y) of the centre of pressure above the water level in the inlet well

= R.L. of C.P. – R.L. of water level in inlet well

= (Known)

Now using the eqn.

$$V_1 = \sqrt{\frac{gX^2}{2.Y}}^{*} \quad ...(12.7)$$

where X and Y are the coordinates of the jet (issuing from centre of pressure) w.r.t. the water surface level in the inlet well.

---

*The eqn. (12.7) can be derived as below w.r.t. to Fig. 12.12 :

$X = V_1 t$ (after a time t)

$Y = \frac{1}{2}gt^2$ $\left[\because \text{using } S = ut + \frac{1}{2}gt^2 \text{ and } u = 0, \text{ we have } S = \frac{1}{2}gt^2\right]$

or $Y = \frac{1}{2}g \cdot \left(\frac{X}{V_1}\right)^2$

or $V_1 = \sqrt{\frac{gX^2}{2y}}$

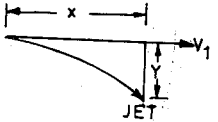


Fig. 12.12

The value of X can be determined. Finally the dia of the inlet well may be kept at about 1.5 times the value of X. The entire procedure will become more clear when we solve the following numerical example.

Example 12.2. *Design the salient dimensions of a syphon well drop for the following particulars :*

Fall	$= 3.8\ m$
General ground level	$= + 163.36\ m$
Full supply depth	$= 75\ cm$
Bed level upstream	$= + 162.83$
Discharge	$= 1\ cumec$
Bed width upstream and downstream	$= 2.4\ m$

(Madras University 1972)

Solution. For a trapezoidal notch, we have the discharge eqn. (12.4) as

$$Q = 2.22 \cdot H^{3/2} [l + 0.4\, n\, H]$$

At full supply discharge, we have

$$Q_{100} = 2.22\, (y_{100})^{3/2} [l + 0.4\, n\, y_{100}]$$

where $y_{100} = F.S.D. = 0.75$ m

$Q_{100} = F.S.Q = 1$ cumec

$$\therefore \quad 1 = 2.22\, (0.75)^{3/2} [l + 0.4n\, (0.75)]$$

or $$0.71 = l + 0.3\, n \qquad ...(i)$$

At 50% full discharge, we have

$$Q_{50} = 2.22\, y_{50}^{3/2} [l + 0.4n\, y_{50}]$$

where $y_{50} \approx 0.66\, y_{100}$ (eqn. 12.5)

$= 0.66 \times 0.75$

$= 0.5$ m.

$Q_{50} = 0.5$ cumec

$$\therefore \quad 0.5 = 2.22\, (0.5)^{3/2} [l + 0.4n\, (0.5)]$$

or $$0.64 = l + 0.2\, n \qquad ...(ii)$$

Subtracting (ii) from (i) we get

$$0.07 = 0.1\, n$$

or $$n = 0.7$$

$$\therefore \quad 2 \tan \frac{\alpha}{2} = 0.7, \text{ or } \frac{\alpha}{2} = 19.3°$$

Substituting this value of n in (ii), we get

$$l = 0.64 - 0.14 = 0.50$$

Hence, provide a trapezoidal notch in the steining of the inlet well, with 0.5 m bottom width and each side inclined to an angle of 19.3° with the vertical.

Now, the width of water (at FSL) flowing over notch

$$= 0.5 + 0.7 \times (0.75) = 0.5 + 0.525 = 1.025 \text{ m.}$$

Velocity (V_1) over the notch

$$= \frac{\text{F.S.Q.}}{\text{Area of flow over the notch}}$$

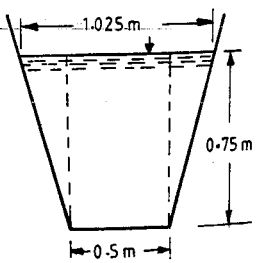


Fig. 12.13

$$= \frac{1}{\frac{0.5+1.025}{2} \times 0.75} \text{ m/sec} = \frac{1}{0.76 \times 0.75} \text{ m/sec} = 1.75 \text{ m/sec}$$

Let us now assume that the diameter of the pipe used be 1 m

$\therefore$ Velocity V_3 through the pipe

$$= \frac{1}{\frac{\pi}{4}(1)^2} \text{ m/sec.} = 1.27 \text{ m/sec.}$$

Let us assume that the diameter of the opening at the inlet of pipe be 0.5 m

$\therefore$ The velocity of entry into the pipe (V_2)

$$= \frac{1}{\frac{\pi}{4}(0.5)^2} \text{ m/s} = 5.1 \text{ m/sec.}$$

Now

Loss of head between the inlet well and the d/s FSL is given by Eqn. (12.6).

$$= 0.5 \cdot \frac{V_2^2}{2g} + \frac{(V_2 - V_3)^2}{2g} + \frac{f' L V_3^2}{2gd} + \frac{V_3^2}{2g}$$

Let us assume that the length of the pipe is kept as 12 m and f' = Darcey's coefficient of friction be taken as equal to 0.012, we then have

$$H_{L_1} = 0.5 \times \frac{(5.1)^2}{2 \times 9.81} + \frac{(5.10 - 1.27)^2}{2 \times 9.81} + \frac{0.012 \times 12 \times (1.27)^2}{2 \times 9.81 \times 1.0} + \frac{(1.27)^2}{2 \times 9.81}$$

$$= 0.66 + 0.77 + 0.01 + 0.08 = \mathbf{1.52\ m.}$$

$\therefore$ R.L. of water surface in the inlet well

$$= \text{d/s FSL} + 1.52$$

$$\left[\begin{array}{l}\text{d/s FSL} = \text{u/s FSL} - \text{fall} \\ \qquad = (162.83 + 0.75) - 3.8 = 159.78)\end{array}\right]$$

$$= 159.78 + 1.52 = \mathbf{161.30.}$$

Approximate R.L. of the centre of pressure (C.P.) of the trapezoidal waterway through notch

$$= \text{u/s canal bed level} + \frac{1}{3} \text{FSD}$$

$$= 162.83 + \frac{0.75}{3} = 162.83 + 0.25 = \mathbf{163.08}$$

$\therefore$ Height Y of C.P. above water level in the inlet well

$$= 163.08 - 161.30 = 1.78 \text{ m.}$$

Now using Eqn. (12.7), we have

$$V_1 = \sqrt{\frac{g \cdot X^2}{2Y}}$$

or

$$X = \sqrt{\frac{V_1^2 2Y}{g}}$$

$$= \sqrt{\frac{(1.75)^2 \times 2 \times 1.78}{9.81}} = 1.75 \times 0.6 = 1.05 \text{ m}.$$

Now, the dia. of the inlet well may be kept at about 1.5 *X*, *i.e.* $1.5 \times 1.05 = 1.575$ m, say 1.6 m. Keep the dia of the d/s outlet well, as say 1.2 m. Also provide a water cushion at the bottom of the inlet well. Bed and sides of channel for suitable lengths on the u/s as well as d/s side are protected by dry brick pitching. The complete details are shown in Fig. 12.14.

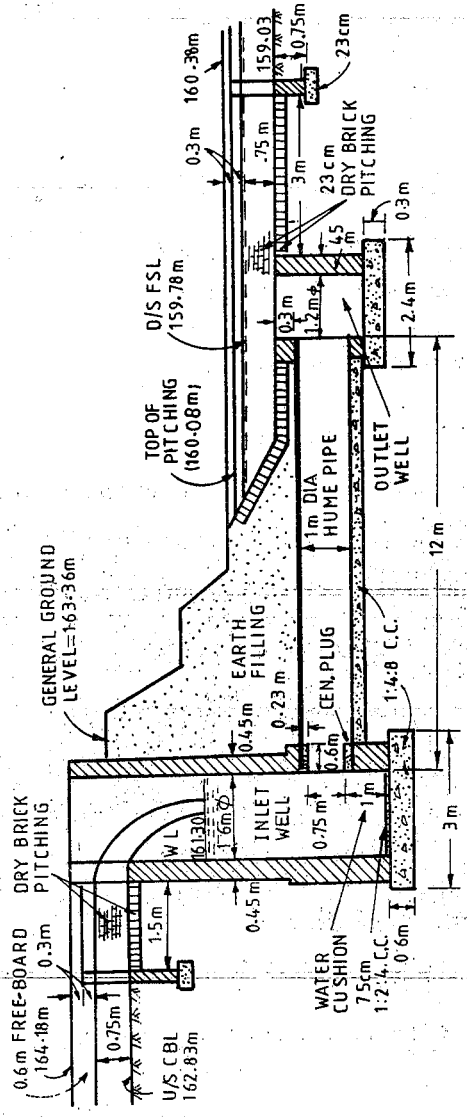


Fig. 12.14

12.5. Design of Simple Vertical drop Fall

In a vertical drop fall, the energy of the flowing water is dissipated by means of impact and by sudden deflection of velocity from vertical to horizontal direction. A water cushion is provided at the toe of the drop, so as to reduce the impact of falling jet and thus to save the downstream floor from scour. The water cushion is formed by depressing the floor below the downstream bed of the canal, as shown in Fig. 12.15.

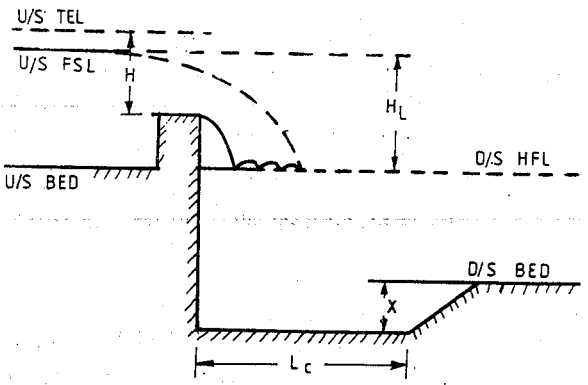


Fig. 12.15

The following dimensions for the cistern have been suggested by U.P. Irrigation Research Institute :

$$L_C = 5 \cdot \sqrt{H \,.\, H_L} \qquad ...(12.8)$$

$$X = \frac{1}{4} \cdot (H \,.\, H_L)^{2/3} \qquad ...(12.9)$$

where L_C = The length of the cistern in metres.

X = Cistern depression below the downstream bed in metres.

H = Head of water over the crest, including velocity head, in metres, *i.e.* = (u/s TEL – Crest level).

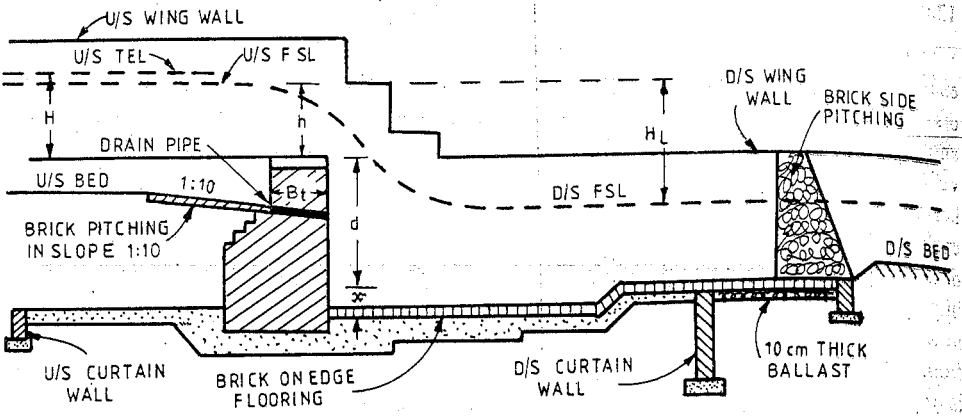
12.6. Design of a Sarda Type Fall

The design criteria for various components of such a fall, based on the recommendations of Bahadarabad Research Station, are given below :

Length of the Crest. Since fluming is not permissible in this type of falls, *the length of the crest is kept equal to the bed width of the canal.* Sometimes, for future expansion, the crest length may be kept equal to (bed width + depth).

Shape of the Crest. A rectangular crest with both faces vertical has been suggested for discharges under 14 cumecs. The top width is kept equal to $0.55\sqrt{d}$ and the minimum base width is kept equal to $\frac{h+d}{G}$ (Take $G = 2$ for masonry) where d is the height of the crest above the downstream bed level and h is the head over the crest [See Fig. 12.16 (*a*)].

For discharges over 14 cumecs, a trapezoidal crest with top width equal to $0.55 \cdot \sqrt{H+d}$ with upstream side slope of 1 : 3 and downstream side slope of 1 : 8 is adopted [See Fig. 12.16 (*b*)].



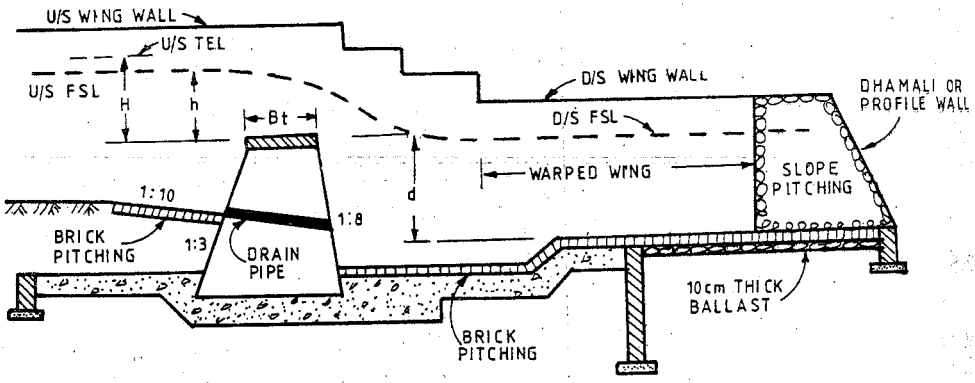
Q = upto a maximum of 14 cumecs

B_t = Top width of crest = $0.55\sqrt{d}$

Base width = $\dfrac{h+d}{2}$

$$Q = 1.84 \cdot LH^{3/2}\left(\frac{H}{B_t}\right)^{1/6}$$

Fig. 12.16. (*a*) Rectangular Crest for Sarda Type fall.



Q = for 14 cumecs and over

B_t = Top width of crest = $0.55\sqrt{H+d}$

Base width = as determined by the batters

$$Q = 1.99 \cdot LH^{3/2}\left(\frac{H}{B_t}\right)^{1/6}$$

Fig. 12.16. (*b*) Trapezoidal crest for a Sarda Type fall.

Crest level. The following discharge formula is used to determine the height of the crest.

$$Q = C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2}\left(\frac{H}{B_t}\right)^{1/6} \qquad ...(12.10)$$

where C_d = 0.415 for rectangular crest

= 0.45 for trapezoidal crest

L = Length of the crest

B_t = Top width of crest.

Height of the crest above bed $= y - h$

$\approx y - H$ (assuming $h \approx H$ *i.e.* neglecting velocity of approach)

where y is the normal depth of channel (upstream).

Upstream Wing Wall. For trapezoidal crest, the upstream wing walls are kept segmental with radius equal to 5 to 6 times H and subtending an angle of 60° at centre, and then carried tangential into the berm as shown in Fig. 12.17. The foundations of the wing walls are laid on the impervious concrete floor itself.

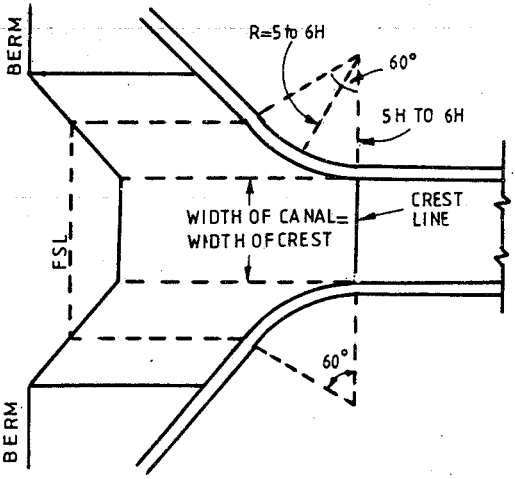


Fig. 12.17. Upstream wing walls for Trapezoidal crest of Sarda Type fall.

For rectangular crest (*i.e.* discharge less than 14 cumecs), the approach wings may be splayed straight at an angle of 45°.

Upstream Protection. Brick pitching in a length equal to upstream water depth may be laid on the upstream bed, sloping towards the crest at a slope of 1 : 10. Drain pipes should also be provided at the u/s bed level in the crest so as to drain out the u/s bed during the closer of the channel.

Upstream Curtain Wall. $1\frac{1}{2}$ brick thick upstream curtain wall is provided, having a depth equal to $\frac{1}{3}$rd of water depth.

Impervious Concrete Floor. The total length of impervious floor can be determined by Bligh's theory for small works and by Khosla's theory for large works. The minimum length of floor on d/s of the toe of the crest wall should be = [2(water depth + 1.2 m) + drop]. The balance can be provided under the crest and on upstream.

The floor thickness required on the downstream side can be worked out for uplift pressures (using minimum thickness of 0.4 m to 0.6 metre) and only a nominal thickness of 0.3 metre is provided on the upstream side. The maximum seepage head will occur when water is stored upto top of crest on u/s side and there is no flow on the downstream side.

Cistern. The length and depth of cistern can be worked out from equations (12.8) and (12.9).

Downstream Protection. The d/s bed may be protected with dry brick pitching, about 20 cm thick resting on 10 cm thick ballast. The length of the d/s pitching is given by the values of Table 12.1; or 3 times the depth of downstream water, whichever is more. The pitching may be provided between two or three curtain walls. The curtain

walls may be $1\frac{1}{2}$ brick thick and of depth equal to $\frac{1}{2}$ the downstream depth; or as given in Table 12.1 (minimum = 0.5 m).

Table 12.1

Head over the crest H (metres)	*Total length of d/s pitching (metres)*	*Remarks*	*Curtain walls*	
			No.	*Depth in metres*
Upto 0.3 m	3.0	All sloping down at 1 in 10	1	0.30
0.3 to 0.45	3.0 + Twice H_L	Horizontal up to end of masonry wings and then sloping down at 1 : 10	1	0.30
0.45 to 0.60	4.5 + Twice H_L		1	0.45
0.60 to 0.75	6.0 + Twice H_L		1	0.60
0.75 to 0.90	9.0 + Twice H_L	"	1	0.75
0.90 to 1.05	13.5 + Twice H_L	"	2	0.90
1.05 to 1.20	18.0 + Twice H_L	"	2	1.05
1.20 to 1.50	22.5 + Twice H_L	"	3	1.35

Slope Pitching. After the return wing, the sides of the channel are pitched with one brick on edge. The pitching should rest on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the downstream water depth. The side pitching may be curtailed at an angle of 45° from the end of the bed pitching, or extended straight from the end of the bed pitching.

Downstream Wings. Downstream wings are kept straight for a length of 5 to 8 time $\sqrt{H \,.\, H_L}$ and may then be gradually wrapped. They should be taken upto the end of the pucca floor.

All wing walls must be designed as retaining walls, subjected to full pressure of submerged soil at their back when the channel is closed. Such a wall generally has a base width equal to $\frac{1}{3}$rd its height.

Example 12.3. *Design a 1.5 metres Sarda type fall for a canal having a discharge of 12 cumecs, with the following data :*

Bed level upstream = *103.0 m*
Side slopes of channel = *1 : 1 m*
Bed level downstream = *101.5 m*
Full supply level upstream = *104.5 m*
Bed width u/s and d/s = *1.0 m*
Soil = *Good loam*
Assume Bligh's Coefficient = *6*

Solution.

Length of crest. Same as d/s bed width = 10 m

Crest level. A rectangular crest is provided, since the discharge is less than 14 cumecs. The discharge formula is given by

$$Q = 1.84 \,.\, L \,.\, H^{3/2} \left[\frac{H}{B_t}\right]^{1/6}$$

Assume top width of the crest as 0.8 m.

$$\therefore \quad 12 = 1.84 \times 10 \times H^{3/2} \times \frac{H^{1/6}}{(0.8)^{1/6}}$$

or $$H^{5/3} = \frac{12 \times 0.964}{1.84 \times 10} = 0.628$$

or $$H = (0.628)^{3/5} = 0.755 \text{ m} \; ; \text{ Say } \mathbf{H = 0.76 \text{ m.}}$$

Velocity of approach

$$= V_a = \frac{\text{Discharge}}{\text{Area}} = \frac{12}{(10 + 1.5)\, 1.5} \qquad (\because \text{Depth of water} = 1.5 \text{ m})$$

$$= \frac{12}{11.5 \times 1.5} = 0.696 \text{ m/sec.}$$

Velocity head $$= \frac{V_a^2}{2g} = 0.025 \text{ m.}$$

u/s TEL = u/s FSL + Velocity Head

$$= 104.5 + 0.025 = 104.525 \text{ m}$$

R.L. of the crest

$$= (\text{u/s TEL} - H)$$

$$= 104.525 - 0.755 = \mathbf{103.77 \text{ m.}}$$

Use crest level of 103.77 metres

Height of the crest above d/s floor

$$= 103.77 - 103.0 = \mathbf{0.77 \text{ m.}}$$

Shape of the crest.

Width of the crest (B_t)

$$= 0.55 \,.\, \sqrt{d}$$

where d = Height of the crest above d/s bed

$$= 103.77 - 101.5 = 2.27 \text{ m}$$

$$\therefore \quad B_t = 0.55 \cdot \sqrt{d} = 0.55 \cdot \sqrt{2.27} = \mathbf{0.825 \text{ m.}}$$

Keep 0.85 m width of the crest

Thickness at base $$= \frac{h + d}{2}$$

$$= \frac{(0.755 - 0.025) + 2.27}{2} = \frac{0.73 + 2.27}{2} = 1.5 \text{ m.}$$

The top shall be capped with 20 cm thick C.C. 1 : 2 : 4

Upstream wing wall. It shall be splayed straight at an angle of 45° from the u/s edge of the crest and shall be embedded by 1.0 m into the berm. On the d/s side, wing walls are kept straight and parallel up to the end of the floor and joined to return walls, as shown in Fig. 12.19.

Upstream protection. 1.5 m long brick pitching (equal to u/s water depth) is laid on the u/s bed, sloping down towards the crest at 1 : 10, and *three drain pipes of 15 cm diameter at the u/s bed level should be provided in the crest so as to drain out the u/s bed during the closure of the canal.*

Upstream curtain wall. Maximum depth of u/s curtain wall

$$= \frac{y_u}{3} = \frac{1.5}{3} = 0.5 \text{ m.}$$

Provide 0.4 m × 0.8 m deep curtain wall on the u/s.

Cistern. Depth of cistern, is given by Eq. 12.9 as

$$X = \frac{1}{4}[H \cdot H_L)^{2/3} = \frac{1}{4}[0.76 \times 1.5]^{2/3} = \frac{1}{4} \times (1.14)^{0.667} \quad \text{...(12.9)}$$

$$= \frac{1}{4} \times 1.091 = 0.273 \text{ m ; } \textbf{Say 0.3 m deep.}$$

∴ R.L. of cistern = 101.5 – 0.3 = **101.2 m.**

Length of cistern = $5\sqrt{H \cdot H_L}$

$= 5 \times \sqrt{0.76 \times 1.5} = 5 \times \sqrt{1.14} = 5.34$ m ; **say 5.5 m.**

Provide 5.5 m long cistern at R.L. 101.2 m.

Impervious floor.

Maximum Static Head

= (Crest level – d/s bed level)

= 103.77 – 101.5 = 2.27 m.

Total floor length required

= *C.H.*; where *C* is Bligh's coefficient

= 6 × 2.27 = 13.62 m. ; say **13.7 m.**

Minimum d/s floor length required

= [2 (Water depth + 1.2) + H_L)

= 2 (1.5 + 1.2) + 1.5 = 2 (2.7) + 1.5 = 5.4 + 1.5 = 6.9 m ; say **7 m.**

Provide 7 m d/s floor and the balance 6.7 m under and upstream of the crest, as shown in Fig. 12.18.

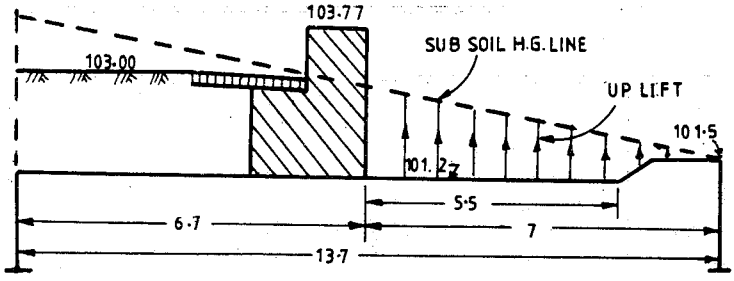


Fig. 12.18

Floor Thicknesses. H.G. line for the maximum static head is shown in Fig. 12.18.

Maximum unbalanced uplift at the d/s toe of the crest

$$= 0.3 + \frac{(103.77 - 101.3)}{13.7} \times 7 = 0.3 + 1.16 = 1.46 \text{ m}$$

Thickness required $\frac{1.46}{1.24} = 1.29$ m ; say 1.3 m.

Provide 1.1 m thick concrete overlain with 0.2 m thick brick pitching.

Unbalanced head at 3 m from the toe of the crest

$$= 0.3 + \frac{2.27}{13.7} \times 4 = 0.3 + 0.67 = 0.97$$

$$\text{Thickness required} = \frac{0.97}{1.24} = 0.78 \text{ m ; say } 0.8 \text{ m.}$$

Use 0.6 m thick concrete with 0.2 m brick layer.

Unbalanced head at 5 m from the toe

$$= 0.3 + \frac{2.27}{13.7} \times 2 = 0.3 + 0.33 = 0.63 \text{ m.}$$

Thickness required

$$= \frac{0.63}{1.24} = 0.51 \text{ m ; Say } 0.55 \text{ m.}$$

Use 0.35 m thick concrete with 20 cm thick brick layer, as shown in Fig. 12.19.

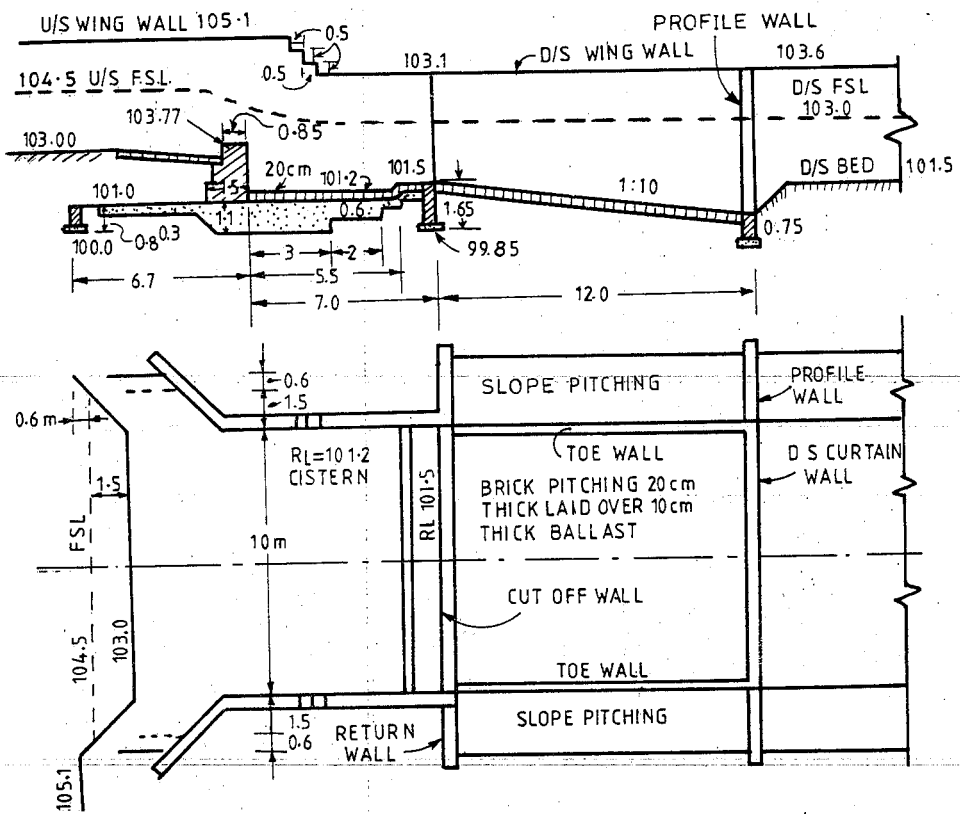


Fig. 12.19. Details of the Sarda Type fall (rectangular crest) of example 12.3.

D/s Curtain Wall. The curtain wall at the d/s end of the floor should be 0.75 m deep (for $H = 0.76$ m in Table 12.1)

Provide 0.4 m × 1.65 m deep curtain wall at d/s end of floor, *i.e.* upto a level of 101.5 – 1.65 = 99.85 metres, *i.e.* the deepest foundation level.

Downstream pitching. From Table 12.1,

Total length of d/s pitching

$$= 9 + 2 \times 1.5 = 12 \text{ metres.}$$

Pitching is kept sloped at 1 : 10. A curtain wall of 0.4 m × 0.75 m shall be provided at the end of the pitching, as shown in Fig. 12.19.

Example 12.4. *Design a 1.5 metres Sarda Type fall for a canal carrying a discharge of 40 cumecs with the following data :*

Bed level upstream	*= 105.0 m.*
Bed level downstream	*= 103.5 m.*
Side slopes of channel	*= 1 : 1*
Full supply level upstream	*= 106.8 m.*
Full supply level downstream	*= 105.3 m.*
Berm level u/s	*= 107.4 m*
Bed width u/s and d/s	*= 30 m.*
Safe exit gradient for Khosla's Theory	*= 1/5.*

Solution.

Length of the crest. Is kept equal to bed width = 30 metres.

Crest level. A trapezoidal crest is provided, since the discharge is more than 14 cumecs.

The discharge formula is given by

$$Q = 1.99\, LH^{3/2}\left(\frac{H}{B_t}\right)^{\frac{1}{6}}$$

Assume $B_t = 1.0$ m.

$$\therefore \quad 40 = 1.99 \times 30 \frac{H^{3/2} \cdot H^{1/6}}{(1)^{1/6}}$$

or $$H^{5/3} = \frac{40}{1.99 \times 30} = 0.796$$

or $$H = (0.796)^{3/5} = 0.862 \text{ m; say } \mathbf{0.865\ m.}$$

Velocity of approach

$$\therefore \quad = V_a = \frac{40}{(30 + 1.8)\ 1.8} = 0.7 \text{ m/sec} \qquad (\because \text{Full supply depth} = 1.8 \text{ m})$$

$$\text{Velocity head} = \frac{(0.7)^2}{2g} = 0.025 \text{ m.}$$

u/s TEL = u/s FSL + Velocity head

$$= 106.8 + 0.025 = \mathbf{106.825\ m.}$$

∴ R.L. of the crest

$$= \text{u/s TEL} - H$$

$$= 106.825 - 0.862 = \mathbf{105.963\ m.}$$

Adopt a crest level of 105.97 m.

Shape of the crest. Adopt crest width at top

$$B_t = 0.55\sqrt{H + d}$$

where $H = 0.865$ m

d = Height of the crest above d/s bed

$= 105.97 - 103.5 = 2.47$ m.

$\therefore \quad B_t = 0.55\sqrt{0.865 + 2.47} = 0.55\sqrt{3.335} = \mathbf{1.0\ m.}$

Adopt a trapezoidal crest **with top width of 1.0 m** and u/s slope 1 : 3 and d/s slope 1 : 8.

Upstream Wing Walls. Radius of the wings should be 5 to 6 times the head over the crest $= 5 \times 0.865 = 4.325$, to $6 \times 0.865 = 5.19$ m. Use 5.0 m radius for the wings. U/s wing walls shall be kept segmental with 5 m radius subtending an angle of 60° at centre and then carried tangentially into the berm.

Downstream Wing Walls. The downstream wings shall be kept straight up to a distance of say $6 \cdot \sqrt{H \cdot H_L}$, *i.e.* $6 \cdot \sqrt{0.865 \times 1.5} = 6.8$ m; say 7 m, and then warped in a slope of 1 : 1 and shall be taken upto the end of pucca floor.

Upstream Protection. Brick pitching equal to u/s water depth *i.e.* 1.8 m is laid on the u/s towards the crest at 1 : 10 slope. Provide 20 cm drain holes in the entire length at 3 m c/c to drain out the u/s bed during the closure of the canal.

Upstream Curtain Wall. The minimum depth of curtain wall $= \frac{1}{3}$rd water depth, *i.e.* $\frac{1}{3} \times 1.8 = 0.6$ m. Provide 0.7 m deep masonry wall over 0.3 m thick concrete.

Thus, provide a curtain wall 0.4 m × 1.0 m deep on the u/s.

Downstream Curtain Wall. Minimum thickness

$$= \frac{\text{Depth}}{2} = \frac{1.8}{2} = 0.9 \text{ m.}$$

or from Table 12.1, it is equal to 0.75 m.

Provide a d/s curtain wall 0.4 m × 1.4 m over 0.3 m cement concrete. Thus, total depth of d/s curtain wall shall be 1.7 m with its bottom level at 101.8 m.

Cistern. Depth of cistern

$$= X = \frac{1}{4}(\text{H} \cdot H_L)^{2/3} \qquad \ldots(12.9)$$

$$\therefore \quad X = \frac{1}{4}\left[(0.865 \times 1.5)^{2/3}\right] = \frac{1}{4} \times 1.19 = 0.3 \text{ m (say)}$$

R.L. of Cistern = *103.5 – 0.3 = 103.2 m.*

Length of cistern $= 5\sqrt{H \cdot H_L} = 5 \cdot \sqrt{0.865 \times 1.5} = 5 \times 1.14 = 5.7$ m.

Provide 5.7 m long Cistern

Total Floor Length and Exit Gradient

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

Maximum static head (H) is caused when water is stored upto the crest level and there is no water d/s.

$H = 105.97 - 103.5 = 2.47$ m.

$d = 1.7$ m (*i.e.* Depth of d/s curtain wall)

$G_E = 1/5$ (given)

$$\therefore \quad \frac{1}{5} = \frac{2.47}{1.7} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or $$\frac{1}{\pi\sqrt{\lambda}} = \frac{1.7}{2.47} \times \frac{1}{5} = 0.137$$

From plate 11.2

$$\alpha = 10$$

or $$\frac{b}{d} = 10$$

or $$b = 10 \times 1.7 = 17 \text{ m.}\ ; \textbf{Use 18 m.}$$

Minimum floor length required on the d/s

$$= 2 \text{ (Water depth } + 1.2) + H_L$$

$$= 2\,(1.8 + 1.2) + 0.865 = 6.865 \text{ m}\ ; \textbf{say 7 m.}$$

Provide the balance length of 11 m under and upstream of the crest, as shown in Fig. 12.20.

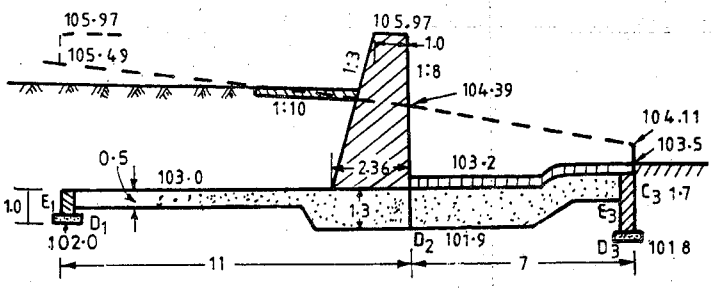


Fig. 12.20

Uplift Pressure Calculations

Assume u/s floor thickness as 0.5 m, and d/s floor thickness as 0.8 m, and floor thickness at toe of the crest as 1.3 m.

(1) *Upstream Wall.*

$$b = 18 \text{ m.}$$

$$d = 1.0 \text{ m.}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.0}{18.0} = 0.056$$

From Plate 11.1 (*a*)

$$\phi_{E_1} = 0\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 16 = 84\%$$

$$\phi_{C_1} = 100 - \phi_E = 100 - 23 = 77\%.$$

Correction to ϕ_{C_1} for depth of floor

$$= \frac{84\% - 77\%}{103.0 - 102.0} \times 0.5 = \frac{7}{1} \times 0.5 = 3.5\% \text{ (+ ve)}$$

Correction due to influence of other wall is very small and neglected.

$$\therefore \quad \phi_{C_1} \textbf{ (corrected)} = 77\% + 3.5\% = \textbf{80.5\%}$$

(2) *Toe of Crest.*

$$b_1 = 11 \text{ m}$$
$$b = 18 \text{ m}$$
$$\frac{b_1}{b} = \frac{11}{18} = 0.61$$
$$d = 103.2 - 101.9 = 1.3 \text{ m}$$
$$\alpha = \frac{b}{d} = \frac{18}{1.3} = 13.9$$

From Plate 11.1(*b*)

$$\phi_{D_2} = 36\%$$

(3) *Downstream Curtain Wall.*

$$d = 1.7 \text{ m}$$
$$b = 18 \text{ m}$$
$$\frac{d}{b} = \frac{1.7}{18} = 0.094$$

From Plate 11.1(*a*)

$$\phi_{E_3} = \phi_E = 29\%$$
$$\phi_{D_3} = \phi_D = 20\%$$
$$\phi_{C_3} = 0\%$$

Correction for depth to ϕ_{E_2}

$$= \frac{29\% - 20\%}{1.7} \times 0.8 = \frac{9}{1.7} \times 0.8 = \mathbf{4.2\%} \text{ (– ve)}$$

ϕ_{E_3} **(corrected)** $= 29 - 4.2 = \mathbf{24.8\%}$

The levels of H.G. line for **maximum static head** are worked out in Table 12.2 and plotted in Fig. 12.20.

Table 12.2

Condition of flow	*u/s W.L. in metres*	*d/s W.L. in metres*	*Head H in metres*	*Height/Elevation of H.G. line above datum*						
				ϕ_{E_1} *100%*	ϕ_{D_1} *84%*	ϕ_{C_1} *80.5%*	ϕ_{D_2} *36%*	ϕ_{E_3} *24.8%*	ϕ_{D_3} *20%*	ϕ_{C_3} *0%*
Maximum static head, *i.e.* Water up to crest level on u/s and no water downstream	105.97	103.50	2.47	2.47 105.97	2.08 105.58	1.99 105.49	0.89 104.39	0.61 104.11	0.50 104.00	0.90 103.5

Floor Thicknesses

Provide a nominal thickness of 0.4 m under u/s floor.

Unbalanced head at d/s toe of glacis $= 104.39 - 103.2 = 1.19$ m

Thickness required $= \frac{1.19}{1.24} = 0.97$ m ; Use 1.2 m.

Provide 1.0 m thick C.C. overlain by 0.2 m thick brick pitching.

Unbalanced head at d/s end of floor $= 104.11 - 103.5 = 0.61$ m.

If the height of the crest works out to be more they 0.4 times the upstream water depth, the fall may be flumed or fluming ratio increased, so as to increase H and to lower the crest.

Hump. For an unflumed non-meter ordinary fall, the bed approach may have a slope of $\frac{1}{2}:1$ ($\frac{1}{2}H:1V$), joined tangentially to the u/s edge of the crest with a radius of $H/2$, as shown in Fig. 12.23.

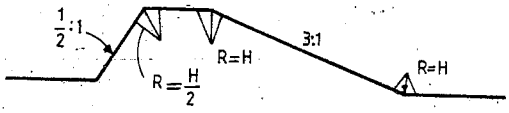


Fig. 12.23

For a flumed meter fall, the curve should start at the same cross-section as the side curves. The radius of hump or bed curve is equal to $\frac{L_a^2+h^2}{2h}$ (see Fig. 12.24).

Upstream Protection. No upstream protection is required for an ordinary unflumed fall. However, for a flumed fall, dry brick pitching on edge may be laid both in bed and sides for a length equal to u/s water depth (in a slope of 1 : 10 in the bed).

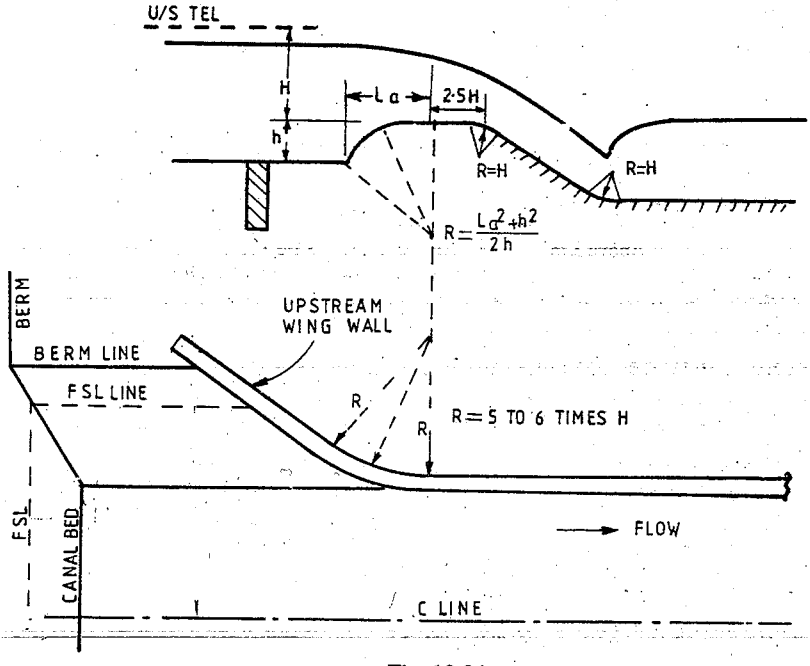


Fig. 12.24

Downstream Glacis. A straight glacis with 2 : 1 slope has been found to be quite suitable for this type of falls. The glacis may be joined to the crest at the u/s end, and to the floor at the d/s end, with a radius = H, as shown in Fig. 12.23.

Length and Thickness of Floor. The floor is designed as per Khosla's theory and the total length of the floor is determined from exit gradient considerations, as was done for Trapezoidal Sarda type fall.

Downstream Protection. With the provision of a deflector wall, no d/s bed pitching is provided, and only side slope pitching is provided for a length equal to 3 times the d/s water depth and should rest on a toe wall.

Upstream Wing Walls. For an unflumed non-meter fall, the side walls may be splayed at 45° from u/s edge of crest and carried into the berm for about one metre [as was done in rectangular crest—Sarda type fall (see Fig. 12.19)].

For a flumed meter fall, curved wings with a radius of 5 to 6 times H subtending an angle of 60° at centre, and therefore, carried tangentially into the berm [as was done in Trapezoidal crest – Sarda type fall (see Fig. 12.17)] may be provided.

Downstream Wing Walls. For an unflumed fall, the walls are taken straight up to the d/s end of the floor and then joined with return walls (as was done in a rectangular crest — Sarda type fall—Fig. 12.19.

For a flumed fall, the d/s wings shall have to be expanded up to the normal width of the river. The expansion can be achieved straight with 1 in 3 slope for smaller works, and a hyperbolic 'expansion (Fig. 12.25), given by equation (12.11), may be used for large works. The hyperbolic expansion is given by equation :

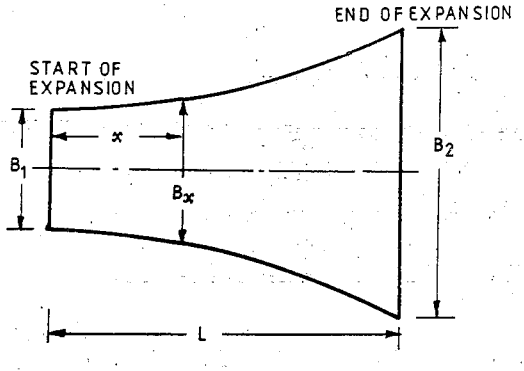


Fig. 12.25. Hyperbolic Expansion.

$$B_x = \frac{B_1 \cdot B_2 \cdot L}{L \cdot B_2 - (B_2 - B_1) \cdot x} \qquad ...(12.11)$$

where B_x is the width of expanding flume at any distance x from the beginning of expansion.

The wings shall be embedded into the berms by at least 1 metre.

Roughening Devices or Energy Dissipators

In Sarda type fall, where energy is dissipated by impact, roughening devices can serve as an additional source of energy dissipation. Similarly, in Glacis fall, roughening devices may be employed to arrest the energy left after jump formation. They may help in jump formation also. Various types of roughening devices used in falls are given below :

(1) **Friction Blocks.** These are the most simple and useful of all such devices. They consist of rectangular concrete blocks securely anchored into the floor. Their height is approximately $\frac{1}{4}$th water depth. The spacing between the blocks is about twice the height of the blocks. The specific recommendations for their use in the modern falls are as given below :

(*a*) *For Vertical Drop Fall or Sarda Type Fall with Trapezoidal crest.* Two rows of friction blocks staggered in plan may be provided, as shown in Fig. 12.26.

The first row may start from a distance 1.5 y_c (where y_c = critical depth) from the d/s toe of the crest. Spacing between the rows is kept equal to y_c.

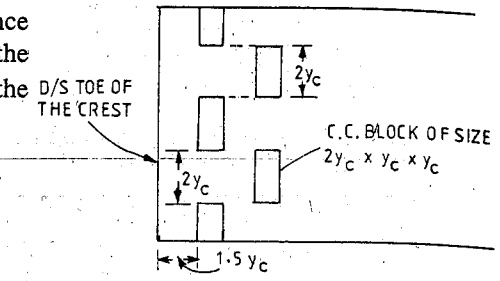


Fig. 12.26. Provision of friction blocks in Sarda type fall.

Spacing between the blocks $= 2 \cdot y_c$.

The dimensions of the blocks

$$= 2y_c \times y_c \times y_c$$

In addition, cube blocks of size $\frac{1}{8}$th to $\frac{1}{16}$th of water depth may be provided at the end of the impervious floor.

(*b*) *For Glacis Fall.* Four rows of staggered blocks are generally provided *in case of flumed falls only. Nothing is provided for unflumed falls.* The first row may be provided at a distance equal to 5 × height of block from the toe of the glacis, as shown in Fig. 12.27. The height of the block is generally kept 1/8th of the water depth. The distance between two rows is equal to h (where h is the height of block).

The dimensions of the blocks are $3h \times h \times h$.

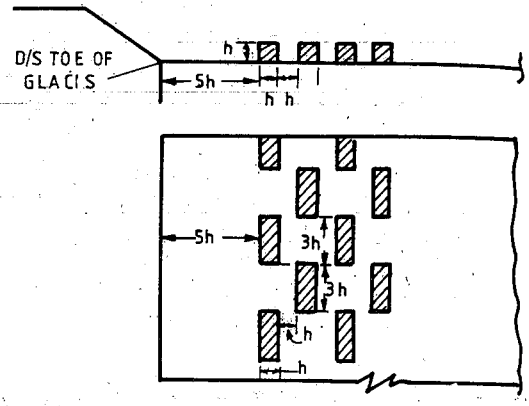


Fig. 12.27. Provision of friction blocks in flumed glacis fall.

(*c*) *For Baffle Fall.* Two rows of friction blocks staggered in plan may be provided only when the drop is more than 2 metres. The suitable dimensions are

Height $(h) = 0.26\, y_2$

where y_2 is the sub-critical depth in the canal d/s, required for formation of jump for parallel downstream sides, in metres.

Length $= h$.

Top width $= \frac{2}{3}h$.

Distance between rows $= h$.

The first row may be placed at a distance $\frac{2}{3}$rd of the cistern length from the upstream end of the cistern.

(2) Glacis Blocks. A single row of blocks called Glacis blocks and of the same dimensions as friction blocks may be provided just at the d/s toe of the glacis, in case of flumed falls with drops more than 2 metres, as shown in Fig. 12.28. It helps in reducing turbulence in flow, which in turn, reduces wave wash, thus ensuring uniform flow.

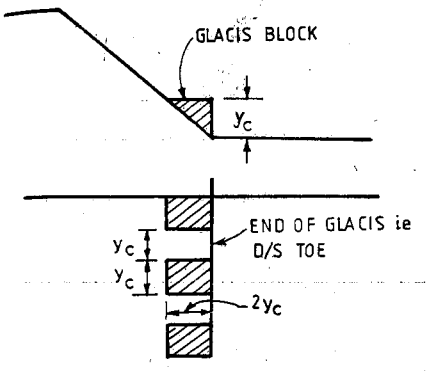


Fig. 12.28. Provision of glacis blocks in flumed falls with drops more than 2 metres.

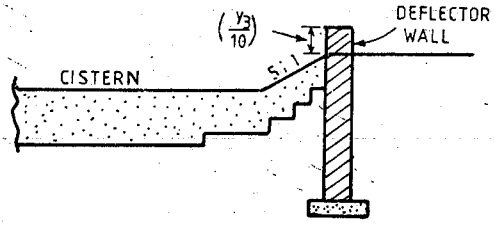


Fig. 12.29. Provision of a deflector wall.

(3) Biff Wall or Deflector Wall. A deflector wall of height approximately $\frac{1}{10}$th of water depth may be provided at the downstream end of concrete floor (in cast with d/s cutoff) as shown in Fig. 12.29.

Example 12.3. *Design a straight flumed meter glacis fall with the following data :*

Full supply discharge of the canal	*= 120 cumecs.*
Bed level of the canal upstream	*= 107.5 m*
Bed level of the canal downstream	*= 106.0 m*
Drop (H_L)	*= 1.5 m.*
FSL of the canal upstream	*= 109.7 m*
FSL of the canal downstream	*= 108.2 m*
Bed width of the canal u/s and d/s	*= 60 m*
Safe Exit Gradient for canal material	$= \frac{1}{5.5}$.

Solution

Length of Crest. From Table 12.3, for $H_L = 1.5$,

fluming ratio = 75%

∴ Length of the crest

= 75% of Bed width of canal

= 75% × 60 m = **45 m.**

Adopt crest length = 45 m.

Crest Level. Since the fall is to be used as a meter, a broad crest shall be provided.

∴ Discharge Q is given by

$$Q = 1.70 LH^{3/2}.$$

∴ $$120 = 1.70 \times 45\, H^{3/2}.$$

or $$H^{3/2} = \frac{120}{1.7 \times 45} = 1.57$$

or $$H = (1.57)^{2/3} = \mathbf{1.35\ m}$$

Velocity of approach

$$= V_a = \frac{120}{(60 + 2.2)\,2.2} = 0.876$$

[assuming 1 : 1 side slopes of canal and taking depth = 2.2 as given]

Velocity head $= \frac{(0.876)^2}{2 \times 9.81} = 0.039$ m; **say 0.04 m.**

u/s TEL = u/s FSL + Velocity head
= 109.7 + 0.04 = 109.74 m.

Crest Level = u/s TEL $- H$
= 109.74 – 1.35 = 108.39 m.

Adopt crest level = 108.39 m.

Height of the crest above u/s bed
= 108.39 – 107.5 = 0.89 m.

which is approximately equal to 0.4 times the upstream water depth ($2.2 \times 0.4 = 0.88$ m) and hence, we can go ahead with this fluming ratio and the present crest level.

Crest Width. Adopt crest width for broad crest
$= 2.5H = 2.5 \times 1.35 = 3.375$ m.

Provide crest width = **3.38 m.**

Hump. The crest shall be joined with upstream bed with a curve of radius

$$R = \frac{L_a^2 + h^2}{2h}$$

where $L_a = 2$ m (assume).
$h = 0.89$ (ht. of crest above u/s bed).

∴ Radius of the curve

$$R = \frac{2^2 + (0.89)^2}{2 \times 0.89} = \frac{4.79}{1.78} = \mathbf{2.69\ m}$$

Cistern. The d/s glacis is provided in a slope of 2 : 1. The cistern level is worked out as below :

$Q = 120$ cumecs.

Width provided = 45 m.

q = Discharge intensity $= \frac{120}{45} = 2.27$ cumecs/metre

$H_L = 1.5$ m.

From plate 10.1, $E_{f_2} = 1.85$ m.

Level at which jump will form

= d/s TEL $- E_{f_2}$ = 108.24 – 1.85 = **106.39 m.**

$1.25\ E_{f_2} = 2.31$ m

R.L. of Cistern = d/s TEL $- 1.25\ E_{f_2}$
= (108.2 + 0.04) – 2.31.
= 108.24 – 2.31 = **105.93 m.**

which is lower than the d/s bed level, and hence, **adopt cistern level = 105.93 m.**

Length of Cistern. 5 to 6 times E_{f_2}, *i.e.* $5 \times 1.85 = 9.25$ m; or $6 \times 1.85 = 11.10$ m. Hence, **use 10 m length of cistern** joined to d/s bed in a slope of 5 : 1 in a length of 5 (106 – 105.93) = 0.35 m.

Downstream Curtain Walls. Depth of d/s curtain wall below bed $= \frac{\text{water depth}}{2}$

$= \frac{2.2}{2} = 1.1$ m; or from Table 10.2, it is equal to 1.35 m, therefore, adopt a curtain wall of total depth equal to 1.4 m, *i.e.* 1.1 m masonry laid over 0.3 m concrete.

Total Floor Length and Exit Gradient. Maximum static head (H) is caused when water is stored up to crest level on the u/s and there is no water d/s.

Maximum static head $(H) = 108.39 - 106.0 = 2.39$ m

Depth of d/s curtain wall $(d) = 1.4$ m.

Safe Exit gradient $G_E = \frac{1}{5.5}$

But $$G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

$$\therefore \quad \frac{1}{5.5} = \frac{2.39}{1.4} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or $$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5.5} \times \frac{1.4}{2.39} = \mathbf{0.107}$$

From Plate 11.2,

$$\alpha = 17$$

Total floor length required

$$= \alpha \cdot d = 17 \times 1.4 = 23.8 \text{ m. ; } \mathbf{say\ 24\ m.}$$

The distribution of floor length is shown in Fig. 12.29. 20.65 m is already provided and the balance of 3.35 m is provided as u/s floor length.

Upstream Curtain Wall. A curtain wall of minimum depth $\frac{y_u}{3} = \frac{2.2}{3} = 0.73$ m must be provided at the end of pucca floor. Let us provide an u/s curtain wall (cutoff) of 0.4 m × 0.8 m deep masonry wall laid over 0.3 m thick cement concrete, thus giving a total depth of cutoff as 1.1 m.

Uplift Pressures. Refer Fig. 12.30. Assume floor thickness on the u/s = 0.3 m and thickness on d/s = 0.8 m

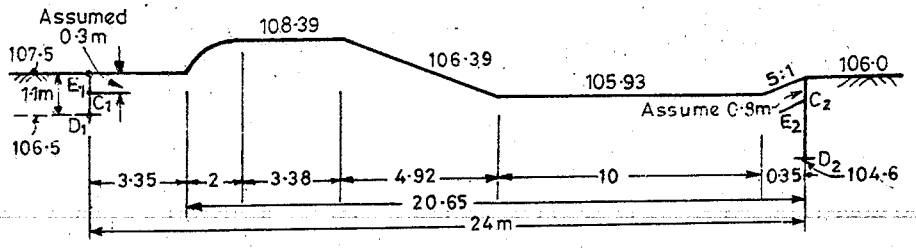


Fig. 12.30

U/s Curtain Wall

$$b = 24 \text{ m}$$

$$d = 1.1 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.1}{24} = 0.046$$

From Plate 11.1 (*a*)

$$\phi_{E_1} = \mathbf{100\%}$$

$$\phi_{C_1} = 100 - \phi_C = 100 - 20 = 80\%$$

$$\phi_{D_1} = 100 - \phi_D = 100 - 13 = \mathbf{87\%}$$

Correction for depth to ϕ_{C_1}

$$= \frac{87\% - 80\%}{1.1} \times 0.3 = 1.91\% \text{ (+ ve)}$$

ϕ_{C_1} **(corrected)** $= 80 + 1.91 = \mathbf{81.91\%}$

D/s Curtain Wall

$$b = 24 \text{ m}$$

$$d = 1.4 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.4}{24} = 0.0583$$

From Plate 11.1 (*a*)

$$\phi_{E_2} = \phi_E = 23\%$$

$$\phi_{D_2} = \phi_D = \mathbf{15\%}$$

$$\phi_{C_2} = \mathbf{0\%}$$

Correction for depth to ϕ_{E2}

$$= \frac{23\% - 15\%}{1.4} \times 0.8 = 4.6\% \text{ (– ve)}$$

ϕ_{E_2} **(corrected)** $= 23\% + 4.6\% = \mathbf{27.6\%}$

The levels of H.G. lines for maximum static head and flow at FSL are worked out in Table 12.4 and plotted in Figs. 12.31 (*a*) and (*b*), respectively.

Table 12.4

Condition of flow	*u/s W.L. in metres*	*d/s W.L. in metres*	*Head in metres*	*Height/elevation of H.G. line above datum*					
				ϕ_{E_1} *100%*	ϕ_{D_1} *87%*	ϕ_{C_1} *81.91%*	ϕ_{E_2} *27.6%*	ϕ_{D_2} *15%*	ϕ_{C_2} *0%*
Maximum Static Head, *i.e.* Water upto Crest level on u/s and no water on d/s	108.39	106.0	2.39	2.39 108.39	2.08 108.08	1.96 107.96	0.61 106.61	0.36 106.36	0 106.0
Flow at FSL	109.7	108.2	1.5	1.5 109.7	1.30 109.50	1.23 109.43	0.41 108.61	0.22 108.42	0 108.2

Floor Thicknesses. Provide 0.3 m thickness under upstream floor.

At Toe of Glacis. Level of H.G. line (*for static head*) at toe of glacis.

$$= 106.61 + \frac{107.96 - 106.61}{24} \times 10.35$$

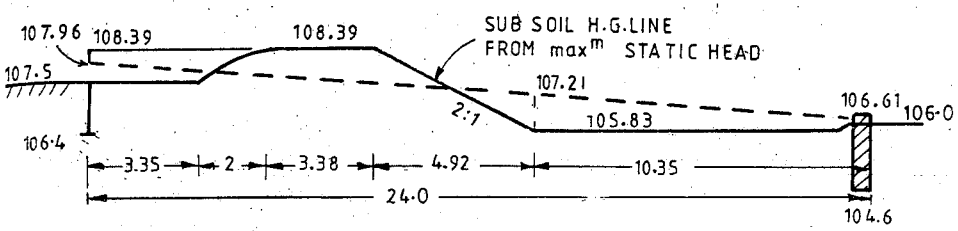


Fig. 12.31. (*a*) Maximum static head condition.

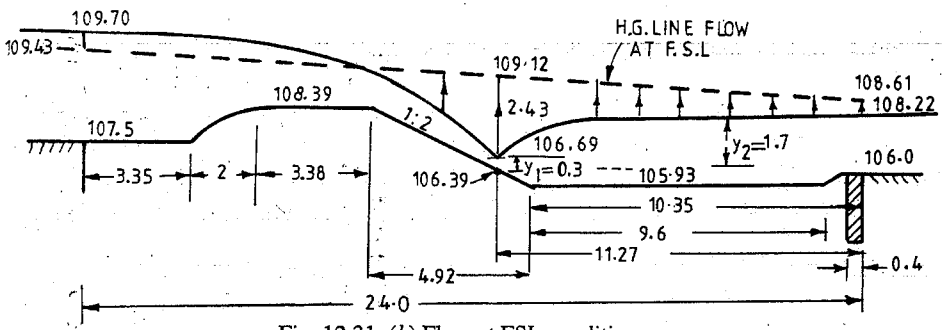


Fig. 12.31. (*b*) Flow at FSL condition.

$$= 106.61 + \frac{1.35}{24} \times 10.35$$

$$= 106.61 + 0.59 = 107.20 \text{ m.}$$

Unbalanced head at the toe due to static head

$$= 107.20 - 105.93 = 1.27 \text{ m}$$

Jump formation

$q = 2.27$ cumecs/metre.

$H_L = 1.5$ m.

$E_{f_2} = 1.85$ m.

$E_{f_1} = E_{f_2} + H_L = 1.85 + 1.50 = 3.35$ m.

From Plate 10.2

y_1 (for E_{f_1}) = 0.3 m.

y_2 (for E_{f_2}) = 1.7 m.

The maximum ordinate at point of jump; from Fig. 12.31 (*b*) = 2.43 m.

$\frac{2}{3}$rd of this ordinate $= \frac{2}{3} \times 2.43 = 1.62$ m, which is more than that for static head.

Hence, the floor thickness at point of jump must not be less than

$$= \frac{1.62}{1.24} = 1.31 \text{ m.}$$

Provide 1.5 m at this point and extend upto 2 m beyond toe

Level of H.G. line at 2 m beyond toe (due to static head)

$$= 106.61 + \frac{1.35}{24} \times 8.35 = 106.61 + 0.47 = \mathbf{107.08\ m.}$$

Unbalanced head = 107.08 – 105.93 = **1.15 m.**

Thickness required = $\frac{1.15}{1.24}$ = 0.925 m., **Provide 1.0 m.**

Level of H.G. line at 5 m beyond toe (due to static head)

$$= 106.61 + \frac{1.35}{24} \times 5.35 = 106.61 + 0.30 = 106.91$$

Unbalanced head due to static head at this point

$$= 106.91 - 105.93 = 0.98 \text{ m.}$$

Thickness required = $\frac{0.98}{1.24}$ = 0.79 m., **Use 0.8 m.**

Upstream Wing Walls. Curved wing walls with radius equal to $6H = 6 \times 1.35 = 8.10$ m subtending an angle of 60° at centre and then carried tangentially into the berm for a suitable length shall be provided as shown in the attached chart Fig. 12.32.

Downstream Expansion. The width of 45 m shall be expanded to 60 m in a splay of 3 : 1.

Minimum length of expansion required = $\frac{3\,(60-45)}{2}$ = **22.5 m.**

The wings can be splayed straight from the d/s toe of the glacis in a length equal to 22.5 m. The bed shall have to be pitched upto the end point of wings. Slope pitching shall be provided in a length $3 \times 2.2 = 6.6$ m beyond this point.

Upstream pitching in a minimum length of 2.2 m (equal to u/s water depth) is required. It shall, however, be provided as shown in Fig. 12.32.

Energy Dissipators. Four rows of staggered friction blocks shall be provided. The height of the block is $\frac{1}{8}$th of water depth, *i.e.* $\frac{1}{8} \times 2.2 = 0.275$ m. The first row shall be at a distance $5 \times 0.275 = 1.375$ m from the d/s toe of glacis. The size of the blocks shall be 0.825 m × 0.275 m × 0.275 m. Distance between rows = 0.275 m.

A deflector wall at end point of floor is provided and is to have a height equal to $\frac{y_d}{10}$, *i.e.* $\frac{2.2}{10}$ = **0.22 m.**

The detailed arrangement is shown in attached chart Fig. 12.32.

12.8. Design of a Baffle Fall or Inglis Fall

Certain flumed type ordinary straight Glacis falls constructed in Punjab were later found to give some serious troubles, which gave rise to the conclusion that considerable surplus energy might remain in water even after the jump formation. One major cause of these troubles was found to be, too rapid expansion after fluming, which may generate cork screw eddies causing deep scours. Research was carried out to eliminate these defects and Baffle fall was evolved.

A baffle fall makes use of the principle of horizontal impact for energy dissipation. The jump is held stable on a horizontal platform by means of a baffle wall (called baffle).

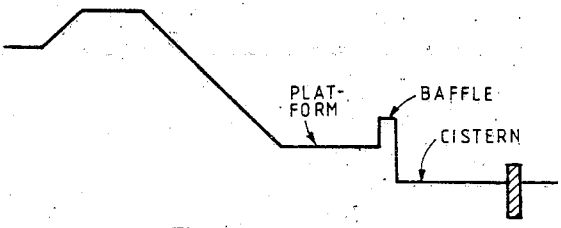


Fig. 12.33. Baffle fall.

Baffle Platform. The horizontal platform is provided at the level at which the jump would normally form. This can be determined by Blench curves in case there is no expansion of the wings in the region of supercritical jet. If the supercritical jet is splayed, the optimum level at which the baffle platform should be provided, can be determined by designer's curves (given in C.B.I. publication No. 10). In the absence of curves, the values can be determined by using the formulas given below.

Subcritical depth (y_2) required for jump formation in ordinary cases without fluming is very nearly given by

$$y_2 = 0.98\, q^{0.52} H_L^{0.21} \qquad \text{...(12.10)}$$

where q = the discharge intensity in cumecs/metre

H_L = Drop in metres.

y_2 = the subcritical depth in metres.

The subcritical depth [y_2 (flumed)] required for jump formation, in case there is a fluming, is given by

$$y_2 \text{ (flumed)} = y_2 + (H_X - H_L) \qquad \text{...(12.11)}$$

where H_X is the calculated drop in metres given by

$$H_X = \frac{H_L}{K^{0.152}} \qquad \text{...(12.12)}$$

where K is the fluming ratio (more than 1), *i.e.*

$$= \frac{\text{Actual width of canal before fluming}}{\text{Flumed width of canal}}$$

The R.L. of the baffle platform will then be given by

(*i*) For unflumed fall = d/s FSL $- y_2$.

(*ii*) For flumed fall = d/s FSL $- y_2$ (flumed)

Baffle Wall. Height of the baffle wall $= h_b = y_c - y_1$ ...(12.13)

where y_c is the critical depth, given by

$$y_c = \left[\frac{q^2}{g}\right]^{1/3} \qquad \text{...(12.14)}$$

and y_1 is prejump super critical depth, given by

$$y_1 = 0.183\, q^{0.89} \cdot H_X^{-0.35} \qquad \text{...(12.15)}$$

$H_X = H_L$, when there is no fluming

$$\textit{Thickness of the baffle wall} = \frac{2}{3} h_b \qquad \text{...(12.16)}$$

$$\textit{Length of baffle platform} = 5.25\, h_b. \qquad \text{...(12.17)}$$

Cistern. A cistern of length equal to 5 y_2 (flumed) (equal to $5 \cdot y_2$, for without fluming) may be provided after the baffle wall. The depth of the cistern below the downstream bed should be 10% of the downstream water depth (y_d), subject to a minimum of 15 cm for distributaries and minors and 30 cm for main canals and branches.

Upstream Wings. The upstream wings can be curved with a radius equal to $3.6\, H^{3/2}$ (where H is the total head over the crest) when H is more than 0.3 m, and equal

to $2H$ when H is less than 0.3 m. The circular wings are continued till they subtend an angle of 60° at the centre, and afterwards they can be extended tangentially for the required length into the berm or bank.

Downstream Wings. The downstream divergence for flumed meter falls can be provided at a slope of 1 in 3 to 1 in 4.

A milder divergence is preferable for straight Glacis fall as well as for Baffle fall (1 in 4 to 1 in 10 depending upon the bed width depth ratio of the downstream channel); but that will make the structure costlier. Hence, a divergence of 1 in 3 is generally used for meter falls of both types. The d/s wings of the unflumed baffle fall are kept as are kept for unflumed straight Glacis fall.

Downstream Glacis. The d/s glacis for unflumed baffle falls is kept at 2/3 : 1; but for flumed meter falls, it is kept as 2 : 1. The glacis is joined to crest at the u/s end and to the floor at the d/s end with a radius equal to H, as was done in the case of a glacis fall.

All other details of pitching, etc. remain the same as for straight glacis fall.

The friction blocks are either not provided (upto 2 m fall); or provided as explained earlier (for drops of more than 2 m.)

Example 12.6. *Design an unflumed non-meter baffle fall for the canal having the following data :*

Full supply discharge	*= 30 cumecs*
Bed level u/s	*= 203.0 m*
Bed level d/s	*= 201.2 m*
FSL u/s	*= 204.3 m*
FSL d/s	*= 202.5 m*
Bed width	*= 28 m*
Drop (H_L)	*= 1.8 m*
Side slopes of channel	*= 1 : 1.*

Solution.

Crest Length. Equal to bed width : Provide 28 m. crest length

Crest level. A sharp narrow crest is provided, for which

$$Q = 1.84\, LH^{3/2}$$

$$\therefore \quad 30 = 1.84 \times 28\, H^{3/2}$$

or $$H^{3/2} = \frac{30}{1.84 \times 28} = 0.582$$

or $$H = (0.582)^{2/3} = 0.697 \text{ m; } \textbf{say 0.7 m.}$$

Velocity of approach

$$= V_a = \frac{30}{(28 + 1.3)\, 1.3} = 0.787 \text{ m/sec.}$$

$$\text{Velocity Head} = \frac{V_a^2}{2g} = \frac{(0.787)^2}{2 \times 9.81} = 0.0315 \text{ m; } \textbf{say 0.03 m.}$$

Now, u/s TEL = u/s FSL + Velocity head

$$= 204.3 + 0.03 = 204.33 \text{ m.}$$

Crest Level = u/s TEL $- H$

$$= 204.33 - 0.7 = 203.63 \text{ m.}$$

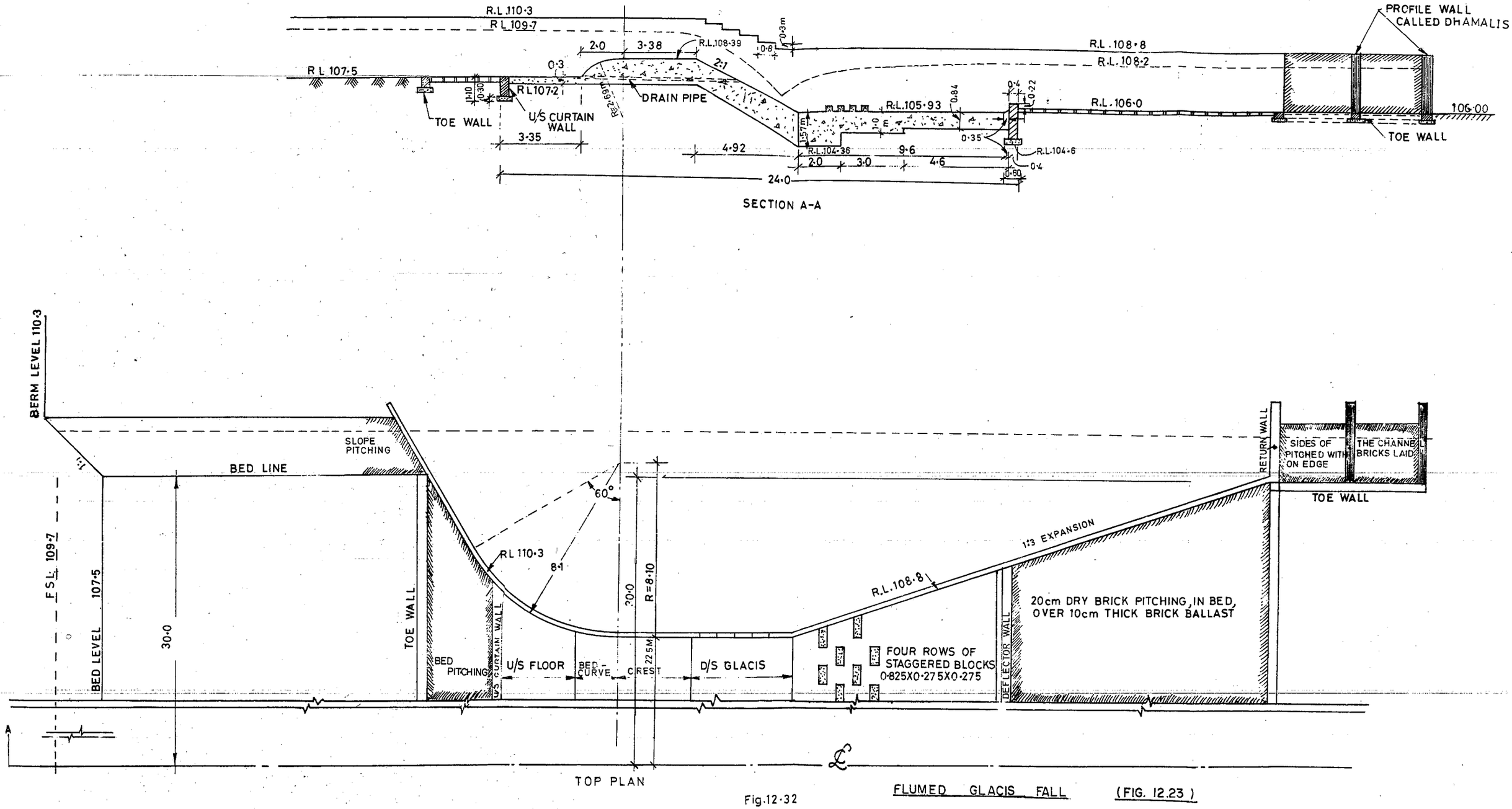


Fig.12·32

FLUMED GLACIS FALL (FIG. 12.23)

Adopt crest level = 203.6 m.

Width of the crest is kept equal to $\frac{2}{3}H = \frac{2}{3} \times 0.7 = 0.47$ m

Provide **crest width = 0.47 m.**

U/s Glacis. Glacis of $\frac{1}{2}$: 1 joined tangentially to the crest with a radius equal to $\frac{H}{2} = 0.35$ m shall be provided.

D/s Glacis. Glacis of $\frac{2}{3}$: 1 joined tangentially to the baffle platform with a radius equal to $H = 0.70$ m shall be provided.

Upstream Wings. The u/s wing walls shall be splayed at an angle of 45° from the u/s end of the floor and shall be embedded into the bank by 1.0 m beyond FSL line.

Downstream Wings. Parallel vertical sides up to the end of pucca floor shall be provided, which shall be connected with the return walls at 90°.

Upstream Protection. No pitching is required in bed and on sides. Depth of u/s curtain wall required is

$$= \frac{y_u}{3} = \frac{1.3}{3} = 0.43 \text{ m.}$$

Provide 0.4 m × 0.6 m deep curtain wall over 0.3 m foundation concrete, thus making its overall depth as 0.9 m.

Baffle Platform and Baffle Wall

Baffle Platform. Since there is no fluming,

$$H_L = H_x = 1.8 \text{ m}$$

Now $y_2 = 0.98 \cdot q^{0.52} \cdot H_L^{0.21}$ (*i.e.* Eq. 12.10)

where $q = \frac{30}{28} = 1.07$ cumecs/metre

$\therefore$ $y_2 = 0.98 \cdot (1.07)^{0.52} \cdot (1.8)^{0.21}$

$= 0.98 \times 1.0358 \times 1.133 = 1.15$ m.

R.L. of baffle platform

$$= \text{d/s FSL} - y_2 = 202.5 - 1.15 = 201.35 \text{ m.}$$

Provide Baffle platform at R.L. = 201.35 m

Baffle Wall. Height of the baffle wall

$$= h_b = y_c - y_1$$

where $y_c = \left[\frac{q^2}{g}\right]^{1/3} = \left[\frac{(1.07)^2}{9.81}\right]^{1/3} = (0.117)^{0.33} = 0.49$ m

$y_1 = 0.183 \cdot q^{0.89} \cdot (H_x)^{-0.35}$ (*i.e.* Eq. 12.15)

$= 0.183\,(1.07)^{0.89}\,(1.8)^{-0.35} = \mathbf{0.16\ m}$

∴ **Height of the baffle wall**

$$h_b = y_c - y_1$$
$$= 0.49 - 0.16 = \mathbf{0.33\ m.}$$

Thickness of baffle wall $= \frac{2}{3} h_b = \mathbf{0.22\ m.}$

Length of baffle platform

$$= 5.25\, h_b = 5.25 \times 0.33 = 1.73 \text{ m. ; } \mathbf{say\ 1.8\ m.}$$

Cistern. Depth of cistern below d/s bed

$$= 0.1\, y_d = 0.1 \times 1.3 = 0.13 \text{ m.}$$

R.L. of cistern $= 201.2 - 0.13 = \mathbf{201.07\ m.}$

Length of cistern $= 5 \cdot y_2$

$$= 5 \times 1.15 = 5.75 \text{ m; } \mathbf{say\ 5.8\ m.}$$

D/s Curtain Wall. Depth of the downstream curtain wall required

$$= \frac{y_d}{2} = \frac{1.3}{2} = 0.65 \text{ m;}$$

or from Table 12.1, the depth for the curtain wall is equal to 0.6 m.

Provide 0.4 m × 1.0 m deep curtain wall, over 0.3 m thick foundation concrete, thus making a total depth of curtain wall = 1.3 m

Height of deflector above d/s bed

$$= \frac{y_d}{10} = \frac{1.3}{10} = 0.13 \text{ m.}$$

Hence, the d/s curtain wall shall be raised by 0.13 m above d/s bed.

D/s Bed Pitching. No pitching is required.

D/s Side Slope Pitching. Is required in a length equal to

$$3 \cdot y_d = 3 \times 1.3 = 3.9 \text{ m.}$$

Provide 0.2 m thick dry brick pitching over 0.1 m thick brick ballast in a length equal to 3.9 m. The slope pitching shall rest on a toe wall 0.4 m thick and 0.8 m deep (overall) constructed in the bed at the junction of bed and sides. A solid profile wall called 'Dhamali' shall be constructed at the end of pitching. It shall be 0.4 m thick and plastered in cement mortar.

Friction Blocks. Not required.

Total Floor Length from Exit Gradient Considerations

Safe exit gradient = 1/5

Maximum static head (H) is exerted when water is stored upto crest level on u/s and there is no water on d/s.

∴ $H = 203.6 - 201.2 = 2.4$ m.

Depth of d/s curtain wall $= d = 1.3$ m

Now, $G_E = \frac{H}{d} \cdot \frac{1}{\pi\sqrt{\lambda}}$

$$\frac{1}{5} = \frac{2.4}{1.3} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or $$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{1.3}{2.4} = 0.108$$

From Plate 11.2,

$$\alpha = 16.5$$

$\therefore$ Total floor length required $= \alpha d = 16.5 \times 1.3 = 21.45$ m;

Provide **22 m** overall length.

The floor length already provided (12.365 m) is shown in Fig. 12.34 ; the balance, *i.e.* 9.635 m is now provided on the u/s, as shown in Fig. 12.34.

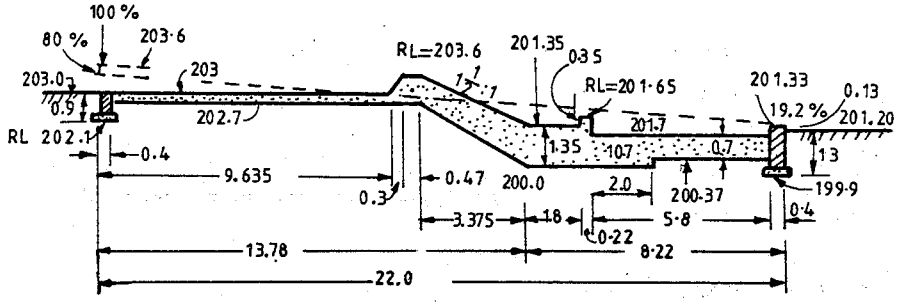


Fig. 12.34. Dimensions of the baffle fall of example 12.6.

Calculations for Uplift Pressures

(*i*) *Upstream curtain wall*

$$b = 22 \text{ m}$$

$$d = 0.9 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{0.9}{22} = 0.041$$

From Plate 11.1 (*a*),

$$\phi_{E_1} = \mathbf{100\%}$$

$$\phi_{D_1} = 100 - \phi_D = 100\% - 14\% = \mathbf{86\%}$$

$$\phi_{C_1} = 100 - \phi_E = 100\% - 20\% = \mathbf{80\%}$$

Assume upstream floor thickness = 0.3 m.

Correction to ϕ_{C_1} *for depth*

$$= \frac{86\% - 80\%}{0.9} \times 0.3 = 2.0\% \text{ (+ ve)}$$

ϕ_{C_1} **(Corrected)** $= 80\% + 2\% = \mathbf{82\%}$

(*ii*) *Downstream curtain wall*

$$b = 22 \text{ m}$$

$$d = 1.3 \text{ m}$$

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.3}{22} = 0.059$$

From Plate 11.1 (*a*)

$$\phi_{E_2} = \phi_E = 23\%$$

$$\phi_{D_2} = \phi_D = \mathbf{16\%}$$
$$\phi_{C_2} = \mathbf{0\%}$$

Assume d/s floor thickness near d/s curtain wall = 0.7 m

Correction to ϕ_{E_2} for floor thickness

$$= \frac{23\% - 16\%}{1.3} \times 0.7 = 3.8\% \text{ (– ve)}$$

ϕ_{E_2} **(Corrected)** $= 23\% - 3.8\% = \mathbf{19.2\%}$.

Uplift pressure at the toe of the glacis

$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 8.22$$
$$= 19.2\% + 22.7\% = 41.9\%$$

Floor Thicknesses

U/s Floor. Provide a nominal thickness of 0.3 m on the upstream side and extend it up to d/s end of crest. Its bottom level shall be at R.L. 202.7 m.

Toe of Glacis

Level of H.G. line at toe of glacis

$$= 201.2 + 41.9\% \times 2.4$$
$$= 201.2 + 1.0 = 202.2 \text{ m}$$

$\therefore$ Unbalanced head due to this maximum static head of 2.4 m

$$= 202.2 - 201.35 = \mathbf{0.85\ m}$$

Unbalanced head due to dynamic condition may be taken as

$$= 50\% (y_2 - y_1) + \% \text{ pressure} \times H_L \qquad \text{(Refer page 558 Chapter 11)}$$
$$= 50\% (1.15 - 0.16) + 41.9\% \times 1.8$$
$$= 0.495 + 0.754 = 1.249 \text{ m ; } \mathbf{say\ 1.25\ m.}$$

Thus, at toe of glacis, the head due to dynamic condition is more than that due to static condition. Hence, minimum thickness required at toe $= \frac{1.25}{1.24} = 1.01$ m. Provide 1.35 m thickness in the entire length of baffle platform, thus keeping its bottom at R.L. 201.0 m.

Thickness at the start of cistern

Percentage pressure at 2.02 m from toe of glacis (*i.e.* start point of cistern)

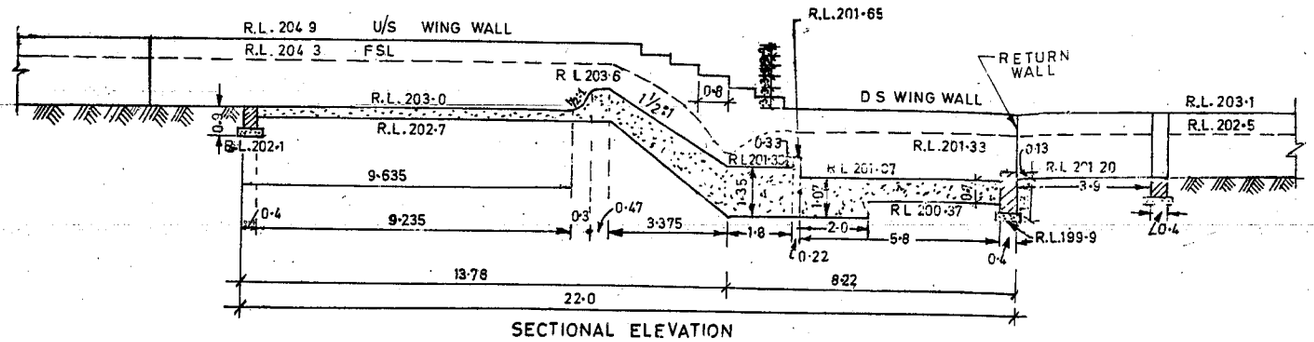
$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 6.2$$
$$= 19.2\% + 17.2 = 36.4\%$$

Level of H.G. line at this point

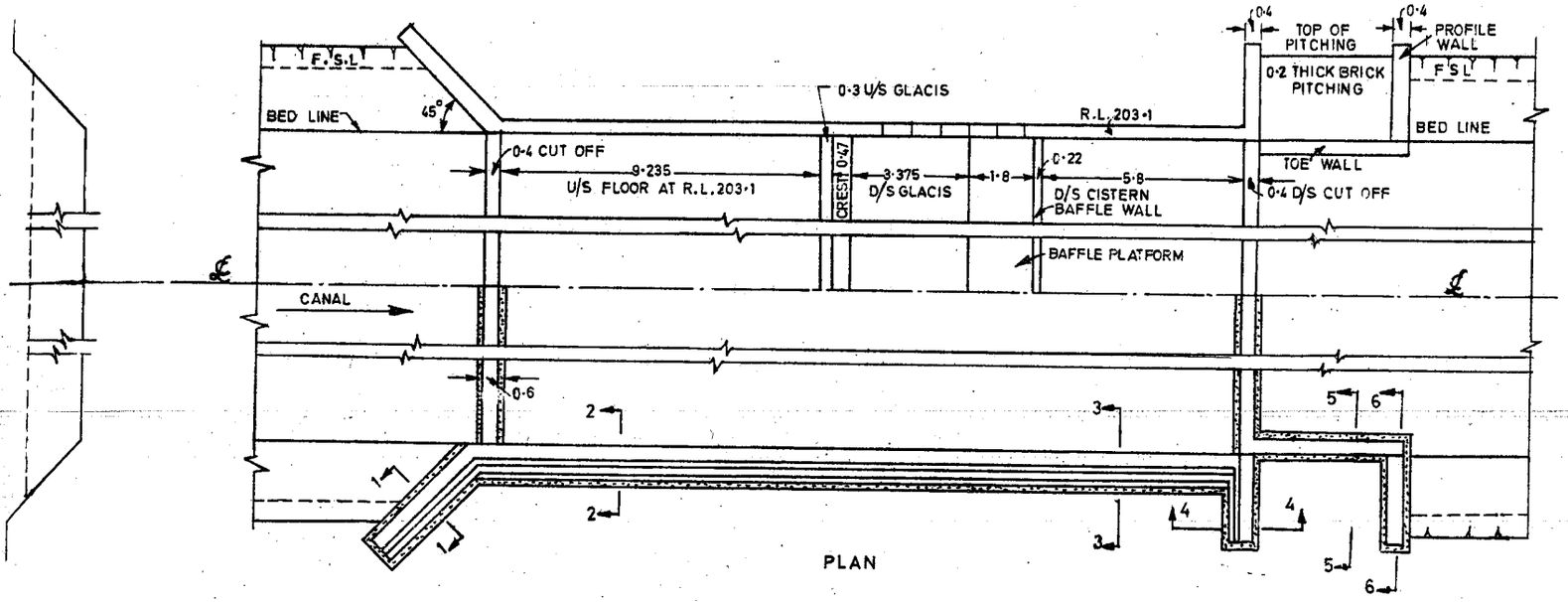
$$= 201.2 + 36.4\% \ (2.4)$$
$$= 201.2 + 0.87 = 202.07 \text{ m.}$$

Maximum unbalanced head at this point

$$= 202.07 - 201.07 = 1.0 \text{ m}$$



SECTIONAL ELEVATION

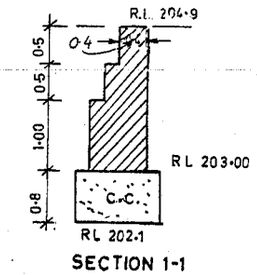


PLAN

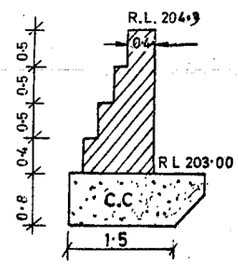
UNFLUMED BAFFLE FALL

Scale 1cm = 1m

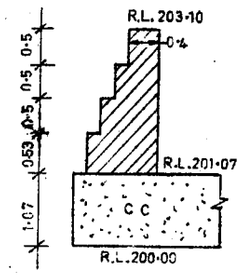
Fig.12·35.



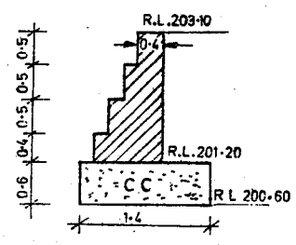
SECTION 1-1



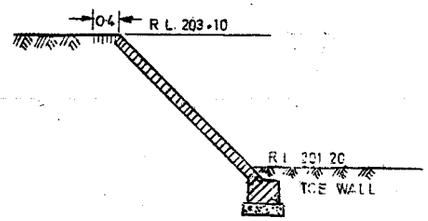
SECTION 2-2



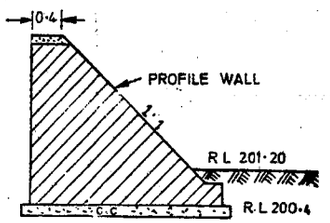
SECTION 3-3



SECTION 4-4



SECTION 5-5



SECTION 6-6

SCALE 1cm=0·5M

Floor thickness required at this point

$$= \frac{1.0}{1.24} = 0.81 \text{ m}$$

Provided thickness $= 201.07 - 200.0 = 1.07$ m.

Thickness required at 3 m beyond the baffle wall

$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 3.2$$

$$= 19.2\% + 8.85\% = 28.05\%$$

Level of H.G. line at this point

$$= 201.2 + 28.05\% \ (2.4)$$

$$= 201.2 + 0.67 = 201.87 \text{ m}$$

Maximum unbalanced head at this point

$$= 201.87 - 201.07 = 0.80 \text{ m}$$

Floor thickness required at this point

$$= \frac{0.80}{1.24} = 0.645 \text{ m}$$

Hence, provide 0.7 m thickness in the remaining portion, as shown in Fig. 12.34. Full details of the fall are shown in the attached chart Fig. 12.35.

PROBLEMS

1. (*a*) What is meant by 'Canal drops' ? Why are canal drops constructed in a canal system ?

(*b*) Enumerate the various types of canal drops which have been used since olden days. Explain in details the design principles governing any one of the modern types.

2. Why are "drops" constructed in an irrigation canal ?

Draw a neat sketch of a syphon well drop and explain briefly its components.

(Madras University, 1973)

3. (*a*) What is meant by "falls" and where are they located ?

(*b*) Discuss briefly the components of various types of falls with neat sketches. Also discuss the suitability of each type.

4. Sketch a syphon well drop to carry 0.3 cumec of water from the following data :

Ground level	= 30.00 m.
B.L. of channel above drop	= 28.00 m.
F.S.L. of channel above drop	= 30.00 m.
F.S.L. of channel below drop	= 28.50 m.
B.L. Of channel below drop	= 26.50 m.

Provide a 4 m cart track over the well drop. The earthwork connections should be clearly shown. Assume any other data that you may require. *(Madras University, 1976)*

5. (*a*) Explain why trapezoidal notches are preferred to rectangular notches in the design of canal drops. *(Madras University, 1974)*

(*b*) Design the size and number of notches required for a canal drop with the following particulars:

Full supply discharge	= 20 cumecs
Bed width	= 14 m.
F.S. depth	= 1.9 m.

Assume any other data if required.

Ans. [6 Notches as with $l = 0.83$ m and $n = 0.37$, provided in a crest raised above the bed level by 0.6 m.]

6. (*a*) Discuss the comparative merits and demerits of Notch falls and Sarda type falls.

(*b*) Design 1 m Sarda type fall on a channel carrying 20 cumecs discharge whose bed width and water depth are 14 m and 1.9 m respectively.

7. (*a*) What are canal falls and why are they constructed ?

(*b*) Design an unflumed non-meter hydraulic jump type fall (*i.e.* glacis fall) on a distributary carrying 10 cumecs discharge. The drop to be affected is 2 m, and the depth of flow and the bed width of channel are 1.4 m and 8.0 m respectively.

8. (*a*) Discuss the principle advantage offered by a baffle fall in comparison to a straight glacis fall.

(*b*) Design a flumed baffle fall for a canal having the following data :

Full supply discharge of the canal	= 120 cumecs.
Bed level of canal upstream	= 107.5 m.
Bed level of canal downstream	= 105.5 m.
Drop (H_L)	= 2 m.
F.S.L. of canal upstream	= 109.7 m
F.S.L. of canal downstream	= 107.7 m.
Bed width of canal u/s and d/s	= 60 m.
Bligh's safe hydraulic gradient for the soil	= C = 12.

9. (*a*) What are "canal falls" and where are they located ?

(*b*) What is meant by "flumed falls" and what are their advantages ?

(*c*) Discuss how fluming is done for flumed falls. Write down the equations etc. that you will adopt for designing the upstream as well as downstream wing walls of flumed falls.

(*d*) What are "roughening devices" ? Discuss their use in 'falls construction' ? What roughening device would you recommend for a straight glacis fall ? (*a*) when flumed (*b*) when not flumed.

10. (*a*) State briefly how you will decide the location of a canal drop. Explain how a trapezoidal notch type fall helps to maintain depth discharge relation in the canal.

(*b*) Following data were observed in a canal fall :

Full supply level	= 115 m.
Bed level of canal	= 112 m.
Bed width of canal	= 15 m.
Full supply discharge	= 30 cumecs
Side slope of canal	= 2 (H) : 1 (V)
Length of crest of the fall	= 10 m (crest section is rectangular)
Coefficient of discharge over crest	= 1.70 $m^{1/2}$/sec.

Calculate the crest level.

[**Solution.** Q = 30 cumecs

y = 3 m.

B = 15 m.

$A = (B + 2y)\, y$ (with 2H : 1 V slopes) = 63 sq. m.

V_a = velocity in channel

$$= \text{velocity of approach} = \frac{Q}{A} = \frac{30}{63} = 0.48$$

$$\frac{V_a^2}{2g} = 0.0117; \textbf{ say } \textbf{0.012 m}$$

$$\therefore \text{ U/s T.E.L.} = \text{u/s F.S.L.} + \frac{V_a^2}{2g} = 115 + 0{:}012 = 115.012 \text{ m.}$$

Now, $Q = 1.7\, L \cdot H^{3/2}$

$30 = 1.7 \times 10\, H^{3/2}$

$H = 1.462$ m.

$\therefore$ Crest level = u/s T.E.L. $- H$

$= 115.012 - 1.462 = 113.65$ m. **Ans.]**

11. Write short notes on any three of the following :

(*i*) Syphon well drop.

(*ii*) Roughening Devices.

(*iii*) Wing wall for flumed falls.

(*iv*) Trapezoidal notch fall.

(*v*) Simple vertical drop type fall.

(*vi*) Sarda type fall.

(*vii*) Straight glacis fall

(*viii*) Baffle or Inglis fall.

(*ix*) Different types of falls and their suitability for a particular project.

13

Regulators, Modules, and Miscellaneous Canal Structures

13.1. Canal Regulation

The water which enters into the main canal from the river has to be divided into different Branches and Distributaries, in accordance with the relative urgency of demand on different channels. This process of distribution is called 'Regulation'. To distribute water effectively, the discharge has to be adjusted to any desired value. This aim is achieved by means of regulators.

13.2. Canal Regulation Works

The works which are constructed in order to control and regulate discharges, depths, velocities etc. in canals, are known as canal regulation works. These structures ensure the efficient functioning of a canal irrigation system, by giving full control upon the canals. The important of these structures are :

(*i*) Canal Falls.

(*ii*) Canal Regulators (Head Regulator and Cross Regulator).

(*iii*) Canal Escapes.

(*iv*) Metering Flumes, etc.

(*v*) Canal Outlets and Modules.

Canal falls have already been described in the previous chapter. Now we shall deal with the remaining structures.

CANAL REGULATORS

13.3. Alignment of the off-taking channel

When a branch channel takes off from the main channel (called Parent Channel), the off-take alignment must be carefully designed.

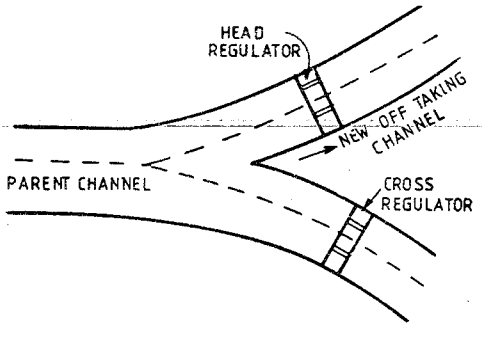


Fig. 13.1

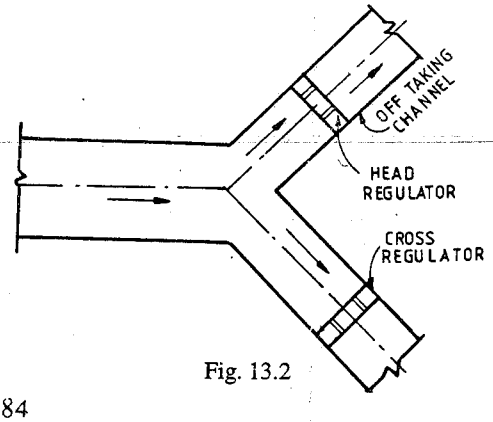


Fig. 13.2

The best ideal alignment is : when the off-taking channel makes zero angle with the parent channel initially and then separates out in a transition, as shown in Fig. 13.1.

The transitions will have to be properly designed, so as to avoid accumulation of silt jetty. As an alternative to the transitions, both the channels should make an angle with the parent channel upstream of the off-take, as shown in Fig. 13.2.

When the parent channel has to be carried straight, the edge of the canal rather than the centre line should be considered in deciding the angle of the off-take (Fig. 13.3).

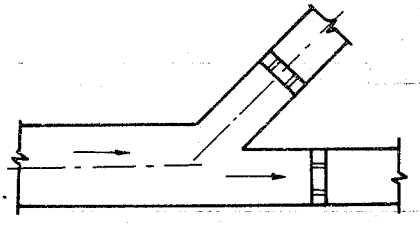


Fig. 13.3

13.4. Distributary Head Regulator and Cross Regulator

A distributary head regulator controls the supply of the off-taking channel ; while a cross-regulator controls the supply of the parent channel.

A head regulator provided at the head of the off-taking channel, controls the flow of water entering this new channel. While a cross-regulator may be required in the main parent channel down-stream of the off-taking channel, and is operated when necessary so as to head up water on its upstream side, thus to ensure the required supply in the off-taking channel even during the periods of low flow in the main channel.

The main functions of a head regulator are :

(*i*) To regulate or control the supplies entering the off-take channel.

(*ii*) To control silt entry into the off-take channel.

(*iii*) To serve as a meter for measuring discharge.

The main functions of a cross-regulator are :

(*i*) To effectively control the entire Canal Irrigation System.

(*ii*) When the water level in the main channel is low, it helps in heading up water on the u/s and to feed the off-take channels to their full demand in rotation.

(*iii*) They help in absorbing fluctuations in various sections of the canal system, and in preventing the possibilities of breaches in the tail reaches.

(*iv*) Cross regulator is often combined with a road bridge, so as to carry the road which may cross the irrigation channel near the site of the cross regulator. It is also usually combined with a fall (if required at the site of cross regulator); when it is called a fall-regulator.

A regulator essentially consists of piers placed across the canal at regular intervals with grooves, in which either planks or gates can be used to control the supplies. Planks called Karries can be used for small channels only, as the maximum height of the planks which can be handled manually is about 2 metres. For large channels, either hand operated or mechanically operated gates can be used.

The hand operated gates can have spans of 6 to 8 m, while mechanically operated gates can be as wide as 20 metres or so. A light bridge platform has to be laid on the piers for operation of planks or gates. It is often economical to combine a regulator with a bridge and also to flume the channel at the site of the regulator, so as to reduce its floor width for affecting economy.

13.5. Design of Cross Regulator and Distributary Head Regulator

Crest Levels. The crest of a cross regulator is generally kept at the upstream bed level of the channel. While, the crest level of the distributary head regulator is generally kept 0.3 to 1.0 m higher than the crest level of the cross regulator.

Water-way. The water-way can be worked out by using the drowned weir formula, given as :

$$Q = \frac{2}{3} \cdot C_{d_1} \sqrt{2g} \cdot B \cdot \left[(h + h_v)^{3/2} - h_v^{3/2} \right] + C_{d_2} \cdot B \cdot h_1 \sqrt{2g\,(h + h_v)} \quad ...(13.1)$$

where $C_{d_1} = 0.577$.

$C_{d_2} = 0.80$.

B = Clear water-way required.

h = Difference of water level u/s and d/s of the crest.

h_1 = Depth of the d/s water level in the channel above the crest.

h_v = Head due to velocity of approach, which is very small and is generally ignored. The discharge formula then becomes.

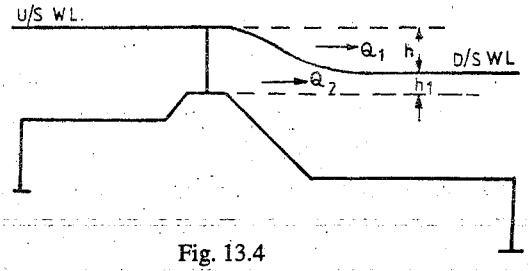


Fig. 13.4

$$Q = \frac{2}{3} \times 0.577 \sqrt{2 \times 9.81}\, B \cdot h^{3/2} + 0.80\, B \cdot h_1 \sqrt{2 \times 9.81} \sqrt{h}$$

$$= 1.69\, B \cdot h^{3/2} + 3.54\, B \cdot h_1 \sqrt{h}$$

or $$Q = B \cdot \sqrt{h} \left[1.69\, h + 3.54\, h_1 \right] \quad ...(13.2)$$

Conditions of flow for design. The design is done for the worst of the following two conditions :

(*a*) Full supply discharge is passing down both the channels with all gates of cross regulator and head regulator fully open.

(*b*) The discharge in the parent channel is low but the off-take channel is running full, and its FSL is maintained by the partial opening of the gates of cross-regulator.

In the first condition, q and H_L are fixed ; while in the second case q reduces and H_L increases, depending upon the low flow of the channel. The first condition of flow generally governs the design, but in certain cases, the second flow condition may become more critical.

If the low flow record of the parent channel is not given, the design can be done for the first flow condition.

Downstream Floor Level or Cistern Level. For the above two flow conditions, q and H_L are worked out. Then E_{f_2} is found from Plate 10.1. The level at which jump would form, *i.e.* the level of d/s floor, is then given by d/s TEL $- E_{f_2}$.

Neglecting velocity head, d/s TEL = d/s FSL

$\therefore$ Level of d/s floor = d/s FSL $- E_{f_2}$.

If the d/s floor for the worst condition works out to be higher than the d/s bed level of the channel, the floor is provided at the bed level itself.

Length of d/s Floor. It is worked out by calculating 5 $(y_2 - y_1)$. If by chance, this length comes out to be small in comparison to 2/3 of total floor length (worked out by exit gradient considerations, *i.e.* $b = \alpha \cdot d$.), then, the length of the d/s floor is kept equal to 2/3 of the total floor length.

Cut-offs are provided as given below :

Upstream Cut-off. The minimum depth of u/s cut off below u/s floor level is kept as $\left(\frac{y_u}{3} + 0.6 \text{ m}\right)$, where y_u is the depth of water in the channel upstream.

Downstream Cut-off. The minimum depth of d/s cut-off below the d/s floor level is kept as $\left(\frac{y_d}{2} + 0.6 \text{ m}\right)$, where y_d is the depth of water in the channel downstream.

Total floor Length. The total floor length 'b' is worked out from safe exit gradient considerations, as explained earlier. This total floor length is then suitably distributed upstream and downstream.

Uplift Pressures and Floor Thicknesses. The thicknesses of the d/s floor required, are worked out by working out the uplift pressures by Khosla's theory. The maximum unbalanced heads at key points are worked out for the maximum static head. The pressure at toe of glacis is also worked out for dynamic condition. The floor thicknesses are designed for the worst case, as explained earlier in the design of weirs and falls. A nominal thickness of 0.3 to 0.5 m is provided on u/s side.

Protection Works. The protection works are designed for a scour depth (D) equal to $\left(\frac{y_u}{3} + 0.6 \text{ m}\right)$ on the upstream, and equal to $\left(\frac{y_d}{2} + 0.6 \text{ m}\right)$ on the downstream. The 'C.C. blocks' and 'inverted filter' are provided in a length approximately equal to 1.5 D. The quantity of stone in launching apron is kept as 2.25 D cu. m/metre.

A typical design of such a regulator is illustrated in the example given below :

Example 13.1. *Design a cross regulator and a head regulator for a channel which takes off from the parent channel with the following data :*

Discharge of parent channel	*= 140 cumecs*
Discharge of distributary	*= 15 cumecs*
FSL of the parent channel, u/s	*= 210.0 m*
FSL of the parent channel, d/s	*= 209.8 m*
Bed width of parent channel, u/s	*= 52 m*
Bed width of parent channel, d/s	*= 46 m*
Depth of water in the parent channel d/s and u/s	*= 2.5 m*

FSL of distributary = 209.1 m
Silt factor = 0.8 m
Assume safe exit-gradient $= \frac{1}{5}$

Solution.

(A) Design of Cross Regulator

Crest Levels. Crest level of cross regulator is kept the same as u/s bed level of parent channel = 210.0 – 2.5 = 207.5 m.

Provide Crest at R.L. 207.5 m.

Waterway $Q = \frac{2}{3} C_{d_1} \sqrt{2g} \cdot B\left[(h+h_v)^{3/2} - h_v^{3/2}\right] + C_{d_2} \cdot B \cdot h_1 \sqrt{2g\,(h+h_v)}$ ...(13.1)

Ignoring velocity head

$$Q = B \cdot \sqrt{h}\left[1.69\,h + 3.54\,h_1\right]. \quad \text{...(13.2)}$$

where B is the clear waterway required.

In this case, h = u/s FSL – d/s FSL
= 210.0 – 209.8 = 0.2 m.
h_1 = d/s FSL – Crest Level
= 209.8 – 207.5 = 2.3 m.

$\therefore \quad 140 = B\sqrt{0.20}\,[1.69 \times 0.2 + 3.54 \times 2.3]$

or $$B = \frac{140}{0.447}\left[\frac{1}{8.488}\right] = 36.9 \text{ m.}$$

Provide 5 bays of 8.0 m each with a clear waterway = 5 × 8.0 = 40.0 m.
Provide 4 piers of 1.5 m width each = 6.0 m.
Thus the overall waterway = 40 + 6 = 46 m.

Downstream Floor Level or Cistern Level

$$q = \frac{140}{40} = 3.5 \text{ cumecs/metre.}$$

$H_L = 0.2$ m.

From Plate 10.1, $E_{f_2} = 1.85$ m.

d/s floor level = d/s FSL – E_{f_2}
= 209.8 – 1.85 = 207.95 m.
d/s bed level = 209.8 – 2.5 = 207.3 m.

which is lower than the calculated d/s floor level. **Hence, provide the cistern or d/s floor at R.L. 207.3 m.**

Length of d/s floor. Length of d/s floor required = 5 $(y_2 - y_1)$

From Plate 10.2,

For $E_{f_2} = 1.85$ m ; $y_2 = 1.8$ m.

For $E_{f_1} = E_{f_2} + H_L = 1.85 + 0.2 = 2.05$ m ; $y_1 = 0.8$ m.

$\therefore \quad 5\,(y_2 - y_1) = 5\,(1.8 - 0.8) = \mathbf{5.0\ m.}$

(subject to the limitation of $\frac{2}{3}$rd of total floor length b).

Vertical cut-offs

Provide *upstream cut-off* for a depth

$$= \frac{y_u}{3} + 0.6 = \frac{2.5}{3} + 0.6 = 1.43 \text{ m}$$

below the floor, *i.e.* the level of its bottom

$$= 207.5 - 1.43 = \mathbf{206.07\ m.}$$

Downstream cutoff. Depth of d/s cutoff below floor level

$$= \frac{y_d}{2} + 0.6 = \frac{2.5}{2} + 0.6 = 1.85 \text{ m.}$$

Hence, the bottom level of d/s cutoff = 207.30 – 1.85 = **205.45 m.**

Total Floor Length from Exit Gradient Considerations

$$G_E = \frac{H}{d} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

where H = Maximum static head, which is caused when there is full water on u/s and there is no water on d/s.

H = u/s FSL – d/s bed level

= 210.0 – 207.3 = 2.7 m.

d = Depth of d/s cutoff = 1.85 m

$G_E = \frac{1}{5}$ (given)

$$\therefore \quad \frac{1}{5} = \frac{2.7}{1.85} \cdot \frac{1}{\pi \cdot \sqrt{\lambda}}$$

or $$\frac{1}{\pi \sqrt{\lambda}} = \frac{1}{5} \times \frac{1.85}{2.7} = 0.137.$$

From Plate 11.2,

For $\frac{1}{\pi \sqrt{\lambda}} = 0.137$, $\alpha = 9$

$\therefore \quad b = \alpha d = 9 \times 1.85 = 16.65$ m ; **say 17 m.**

Minimum d/s floor length required

$$= \frac{2}{3} \cdot b = \frac{2}{3} \times 17 = 11.3 \text{ m; which is greater than 5.0 m, } \textit{i.e.}\ 5\ (y_2 - y_1).$$

Hence, **provide 11.3 m as the d/s floor length.**

Glacis length = 2 × 0.2 = 0.4 m

The balance, *i.e.* 17 – 11.3 – 0.4 = 5.3 m is provided as u/s floor length, as shown in Fig. 13.5.

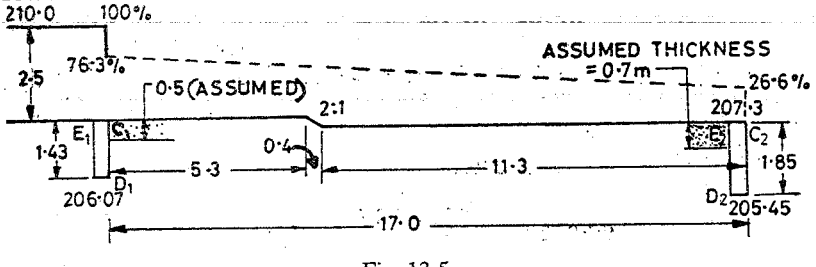


Fig. 13.5

Calculations for Uplift Pressures

Upstream Cut-off

$d = 1.43$ m.

$b = 17.0$ m.

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.43}{17.0} = 0.085$$

From Plate 11.1 (*a*),

$$\phi_{E_1} = 100\%$$

$$\phi_{D_1} = 100\% - \phi_D = 100\% - 18.5\% = 81.5\%$$

$$\phi_{C_1} = 100\% - \phi_E = 100\% - 26.5\% = 73.5\%$$

(Assume u/s floor thickness = 0.5 m)

Correction to ϕ_{C_1} for floor thickness

$$= \frac{81.5 - 73.5}{1.43} \times 0.5 = 2.8\% \text{ (+ ve).}$$

$\therefore$ ϕ_{C_1} **(corrected)** $= 73.5 + 2.8 = 76.3\%$

(*Note.* Correction due to pile interference is small and may be neglected).

Downstream Cut-off

$d = 1.85$ m

$b = 17.0$ m.

$$\therefore \quad \frac{1}{\alpha} = \frac{d}{b} = \frac{1.85}{17.0} = 0.109.$$

From Plate 11.1 (*a*),

$$\phi_{E_2} = \phi_E = 30\%$$

$$\phi_{D_2} = \phi_D = 21\%$$

$$\phi_{C_2} = 0\%$$

(Assume d/s floor thickness near d/s cut off = 0.7 m)

Correction to ϕ_{E_2} due to floor thickness

$$= \frac{30 - 21}{1.85} \times 0.7 = 3.4\% \text{ (– ve)}$$

$\therefore$ ϕ_{E_2} **(corrected)** $= 30\% - 3.4\% = 26.6\%$

(*Note.* Correction due to interference of u/s cut off is neglected).

Floor Thicknesses : D/s Floor

At Toe of Glacis

% pressure at toe of glacis

$$= 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 11.3 = 26.6\% + 33.1\% = 59.7\%$$

Maximum unbalanced head at toe of glacis due to maximum static head (2.7 m)

$$= 59.7\% \times 2.7 = 1.61 \text{ m.}$$

Head due to dynamic action can be taken as

$$50\% (y_2 - y_1) + \phi \, . \, H_L$$

$$= 50\% (1.8 - 0.8) + 59.7\% \times 0.2 = 0.5 + 0.12 = 0.62 \text{ m} < 1.61 \text{ m.}$$

Hence, static head governs the thicknesses.

∴ Thickness required at toe of glacis

$= \frac{1.61}{1.24} = 1.3$ m; **Provide 1.4 m.**

At 3 m beyond toe of glacis

% pressure

$= 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 8.3$

$= 26.6\% + 24.3\% = 50.9\%$

Maximum unbalanced head

$= 50.9\% \times 2.7 = 1.32$ m.

Thickness required

$= \frac{1.32}{1.24} = 1.07$ m; **Provide 1.1 m.**

At 8 m beyond toe of glacis

% pressure

$= 26.6\% + \frac{76.3\% - 26.6\%}{17} \times 3.3$

$= 26.6\% + 9.65\% = 36.25\%$

Maximum unbalanced head

$= 36.25\% \times 2.7 = 0.98$ m.

Thickness required

$= \frac{0.98}{1.24} = 0.79$ m; **Provide 0.8 m**

from this point to the end, as shown in Fig. 13.6.

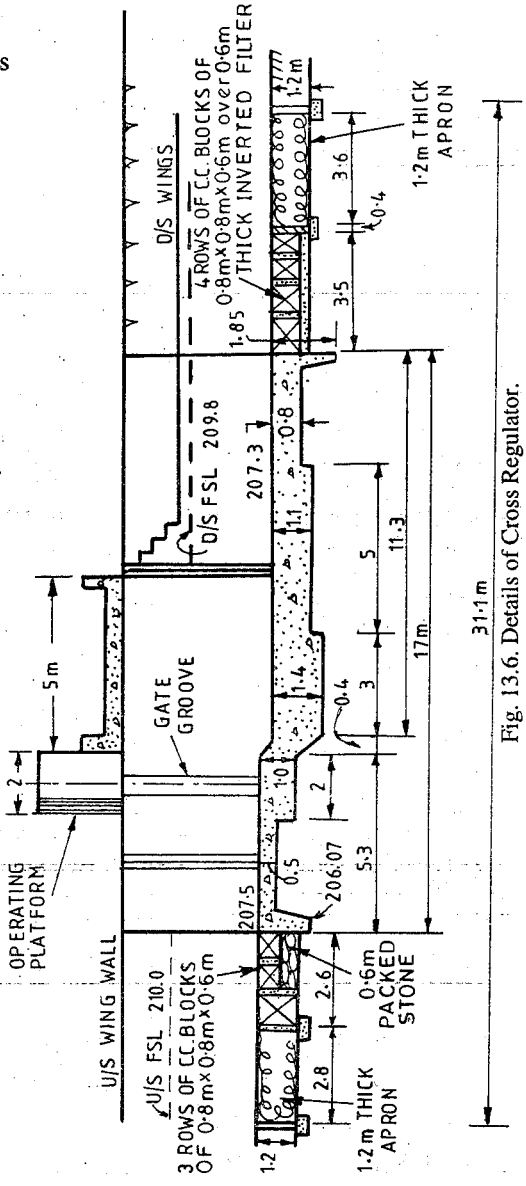


Fig. 13.6. Details of Cross Regulator.

U/s Floor. Theoretically no floor thickness is required under the upstream floor, since the uplift is more than counterbalanced by the weight of the water standing over it. But a nominal thickness of 0.5 m is provided. The floor is thickened to 1.0 m under the crest in a length equal to 2.0 m as shown in Fig. 13.6.

Upstream Protection shall be provided for a scour depth

$$= \frac{y_u}{3} + 0.6 \text{ m} = \frac{2.5}{3} + 0.6 = 1.43 \text{ m}.$$

Launching apron. Provide launching apron of thickness 1.2 m

in a length $= \frac{2.25 \times 1.43}{1.2} = 2.68$ m (∵ Volume required = 2.25 D cu. m/metre)

Use launching apron = 1.2 m thick laid in a length of 2.8 m.

C.C. Blocks. Let us provide C.C. blocks of size 0.8 m × 0.8 m × 0.6 m for a length $D = 1.5 \times 1.43 = 2.2$ m. Hence, 3 rows of C.C. blocks of size 0.8 m × 0.8 m × 0.6 m size with 10 cm. jhories filled with bajri, laid over packed stone apron of 0.6 m thickness, in a total length of 2.6 m, shall be provided as shown in Fig. 13.6.

Downstream Protection shall be provided for a scour depth

$$D = \frac{y_d}{2} + 0.6 = \frac{2.5}{2} + 0.6 = 1.85 \text{ m.}$$

Launching apron. Provide a launching apron of thickness equal to 1.2 m.

Length of apron required

$$= \frac{2.25\,D}{t} = \frac{2.25 \times 1.85}{1.2} = 3.48 \text{ m; } \textbf{say 3.6 m.}$$

Provide a launching apron of 1.2 m thickness laid in a length of 3.6 m.

Inverted filter. Length of filter required = 1.5 × 1.85 = 2.77 m.

Provide 4 rows of C.C. blocks of size 0.8 m × 0.8 m × 0.6 m with 10 cm gaps in between, laid over an inverted filter of 0.6 m thickness, in a total length of 3.5 m, as shown in Fig. 13.6.

Provide 0.4 m thick and 1.2 m deep toe walls between blocks and apron on u/s as well as on d/s.

(B) Design of Distributary Head Regulator

We shall first of all determine a suitable regime section for the distributary, so as to carry the given discharge of 16 cumecs with the given silt factor = 0.8.

From Lacey's Regime Diagrams

Bed width of such a channel = 15.0 m.

Depth of water in this channel = 1.5 m.

Bed Level of Distributaryy = FSL − Depth = 209.1 − 1.5 = **207.6 m.**

Crest Level. The crest level of the Distributary Head is generally kept 0.3 to 1.0 m higher than the bed level of the parent channel. Let us keep it 0.6 m higher.

∴ Crest Level = 207.5 + 0.6 = 208.1 m.

Hence keep the crest of Head Regulator at RL 208.1 m.

Waterway. The discharge is given by

$Q = B \cdot \sqrt{h}\,[1.69\,h + 3.54\,h_1]$ *i.e.* Eq. (13.2)

where h = FSL of parent channel − FSL of Distributary

= 210.0 − 209.1 = 0.9 m.

h_1 = FSL of Distributary − Crest Level

= 209.1 − 208.1 m = 1.0 m.

Q = 15 cumecs.

B = Clear waterway required.

$$\therefore\ 15 = B \cdot \sqrt{0.90}\left[1.69 \times 0.9 + 3.54 \times \sqrt{1.0}\right]$$

or $$15 = B \times 0.949\,[1.521 + 3.54]$$

or $$B = \frac{15}{0.949 \times 5.061} = \textbf{3.12 m.}$$

Which is very small as compared to bed width of distributary, *i.e.* 15.4 m.

Hence, provide 2 bays of 3.0 m each with 1.0 m thick pier in between.

Overall waterway provided = 6 + 1 = 7.0 m

Clear waterway provided = 6.0 m.

The wing walls shall be expanded with proper divergence, so as to provide the normal width of the channel.

Cistern or d/s floor. The discharge intensity q

$$= \frac{15}{6} = 2.5 \text{ cumecs/metre}$$

$$H_L = \text{u/s FSL} - \text{d/s FSL}$$

$$= \text{FSL of parent channel} - \text{FSL of Distributary}$$

$$= 210.0 - 209.1 = 0.9 \text{ m}$$

From plate 10.1,

$$E_{f_2} = 1.8 \text{ m}$$

$$\therefore \quad E_{f_1} = E_{f_2} + H_L = 1.8 + 0.9 = 2.7 \text{ m}.$$

From plate 10.2,

For $E_{f_1} = 2.7$ m ; $y_1 = 0.4$ m

For $E_{f_2} = 1.8$ m ; $y_2 = 1.7$ m.

R.L. of cistern (*i.e.* d/s floor)

$$= \text{d/s FSL} - E_{f_2} = 209.1 - 1.8 = \mathbf{207.3 \text{ m}}.$$

Length of cistern required

$$= 5\,(y_2 - y_1) = 5\,(1.7 - 0.4)$$

$$= 8.0 \text{ m (subjected to a minimum of } \tfrac{2}{3}\, b)$$

Vertical Cutoff. Depth of u/s cutoff below floor

$$= \frac{y_u}{3} + 0.6 = \frac{2.5}{3} + 0.6 = 1.43 \text{ m}.$$

The bottom level of u/s cutoff

$$= 207.5 - 1.43 = \mathbf{206.07 \text{ m}}.$$

The depth of d/s cutoff below floor

$$= \frac{y_d}{2} + 0.6 = \frac{1.5}{2} + 0.6 = 1.35 \text{ m}.$$

Provide d/s cutoff depth = 1.6 m

The bottom of d/s cut-off

$$= 207.3 - 1.6 = \mathbf{205.7 \text{ m}}.$$

Total Floor Length from Exit Gradient Considerations

Maximum static head (H) is caused when there is full water on u/s and no water on d/s

$$\therefore \quad H = 210.0 - 207.3 = 2.7 \text{ m}.$$

d = Depth of d/s cutoff = 1.6 m.

$$G_E = \frac{H}{d}\,\frac{1}{\pi \cdot \sqrt{\lambda}}$$

$$\therefore \quad \frac{1}{5} = \frac{2.7}{1.6} \cdot \frac{1}{\pi\sqrt{\lambda}}$$

or $$\frac{1}{\pi\sqrt{\lambda}} = \frac{1}{5} \times \frac{1.6}{2.7} = 0.119$$

From Plate 11.2,

For $\frac{1}{\pi\sqrt{\lambda}} = 0.119$; $\alpha = 13$

$\therefore \quad b = \alpha \cdot d = 13 \times 1.6 = 20.8$ m; **say 21 m.**

Minimum length of d/s floor required

$= \frac{2}{3} \times 21 = \mathbf{14\ m.}$ (which is greater than 8 m.)

Hence, Provide d/s floor length = 14 m.

Length of d/s Glacis = 1.6 m.
Length of Crest = 1.0 m.
Length of u/s Glacis = 0.6

Total = 17.2 m.

The remaining 3.8 m is provided on the u/s side, as shown in Fig. 13.7.

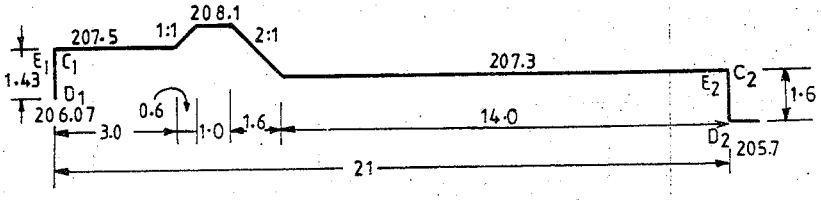


Fig. 13.7

Calculation for Uplift Pressures

(*i*) *Upstream cutoff*

$b = 21$ m.
$d = 1.43$ m.

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.43}{21} = 0.068$$

From Plate 11.1 (*a*),

$\phi_{E_1} = 100\%$

$\phi_{D_1} = 100\% - \phi_D = 100\% - 16.5\% = 83.5\%$

$\phi_{C_1} = 100\% - \phi_E = 100\% - 24\% = 76\%$

Assume u/s floor thickness near cutoff = 0.5 m.

Correction to ϕ_{C_1} due to floor thickness

$$= \frac{83.5\% - 76\%}{1.43} \times 0.5 = 2.62\%\ (+\text{ve})$$

Correction due to d/s pile is neglected.

ϕ_{C_1} **(corrected)** = 76% + 2.62% = 78.62%; **say 78.6%**

(ii) Downstream cutoff

$b = 21.0$ m.

$d = 1.6$ m.

$$\frac{1}{\alpha} = \frac{d}{b} = \frac{1.6}{21.0} = 0.076.$$

From Plate 11.1 (*a*),

$$\phi_{E_2} = \phi_E = 25.5\%$$

$$\phi_{D_2} = \phi_D = 17.5\%$$

$$\phi_{C_2} = 0\%.$$

Assume d/s floor thickness near cutoff = 0.8 m.

Correction to ϕ_{E_2} due to floor thickness

$$= \frac{25.5\% - 17.5\%}{1.6} \times 0.8 = 4.0\% \text{ (– ve).}$$

Correction due to d/s pile is neglected

ϕ_{E_2} **(corrected)** $= 25.5\% - 4.0\% =$ **21.5%**

Floor Thickness

At toe of d/s Glacis

$$\%\text{age pressure} = 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 14$$

$$= 21.5\% + 38.1\% = 59.6\%$$

Maximum unbalanced head due to static head

$$= 59.6\% \times 2.7 = 1.58 \text{ m.}$$

Head due to dynamic action can be taken

$$= 50\% (y_2 - y_1) + \phi \cdot H_L$$

$$= 50\% (1.7 - 0.4) + 59.6\% \times 0.9$$

$$= 0.65 + 0.54 = 1.19 \text{ m; which is less than } 1.58 \text{ m.}$$

Hence, static head governs the floor thicknesses.

Floor thickness required at toe of glacis

$$= \frac{1.58}{1.24} = 1.28 \text{ m; } \textbf{Provide 1.4 m.}$$

At 3 m beyond toe of d/s glacis

$$\%\text{age pressure} = 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 11$$

$$= 21.5\% + 29.9\% = 51.4\%$$

Unbalanced head $= 51.4\% \times 2.7 = 1.39$ m.

Thickness reqd. $= \frac{1.39}{1.24} = 1.12$ m; **Provide 1.2 m.**

At 6 m beyond toe of d/s Glacis

$$\%\text{age pressure} = 21.5\% + \frac{78.6\% - 21.5\%}{21} \times 8 = 21.5\% + 21.8\% = 43.3\%$$

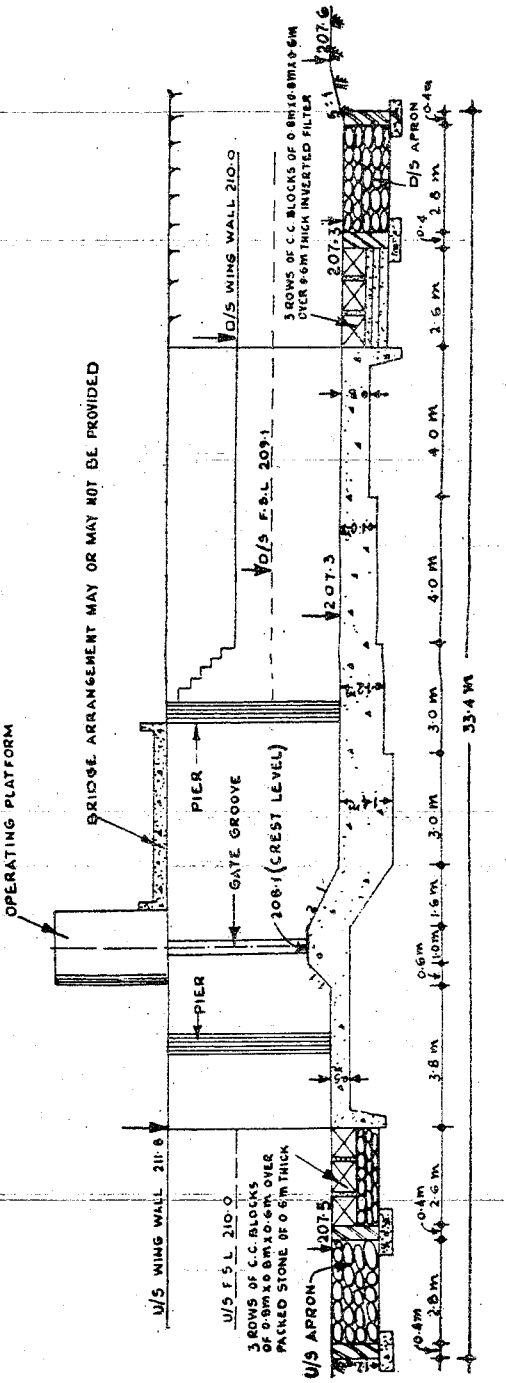


Fig. 13.8. Details of Distributary Head Regulator.

Unbalanced head = $43.3\% \times 2.7 = 1.17$ m

Thickness reqd. = $\frac{1.17}{1.24} = 0.945$ m. **Provide 1 m.**

At 10 m beyond tow of d/s Glacis

%age pressure = $21.5\% + \frac{78.6\% - 21.5\%}{21} \times 4 = 21.5\% + 10.9\% = 32.4\%$

Unbalanced head = $32.4\% \times 2.7 = 0.875$ m.

Thickness reqd. = $\frac{0.875}{1.24} = 0.705$ m; **Provide 0.8 m** upto the end, as shown in Fig. 13.8.

U/s Floor. Provide a nominal thickness of 0.5 m under the u/s floor and extend it under the crest and then join it to the bottom of d/s glacis, as shown in Fig. 13.8.

U/s Protection. Same as provided in the upstream of Cross Regulator.

D/s Protection. The d/s protection shall be provided for a scour depth (D)

$$= \frac{y_d}{2} + 0.6 = \frac{1.5}{2} + 0.6 = \mathbf{1.35\ m.}$$

Launching apron. Provide 1.2 m thick launching stone in a

$$\text{length} = \frac{2.25 \times 1.35}{1.2} = 2.54 \text{ m; } \mathbf{say\ 2.6\ m.}$$

Inverted filter. The length of filter reqd.

$$= 1.5D = 1.5 \times 1.35 = 2.03 \text{ m.}$$

Provide 3 rows of C.C. blocks of 0.8 m × 0.6 m size with 10 cm jhories filled with bajri, laid over 0.6 thick inverted filter.

The details are shown in Fig. 13.8.

CANAL ESCAPES

As the name implies, an escape is a side channel constructed to remove surplus water from an irrigation channel (main canal, branch canal, or distributary, etc.) into a natural drain. The water in the irrigation channel may become surplus due to some mistake; or difficulty in regulation at the head; or due to excessive rainfall in upper reaches. Sometimes, the cultivators may find that the demand of water is over and may close their outlets suddenly. In such circumstances, the canal supplies shall become surplus, and this excess may overflow the banks unless escaped.

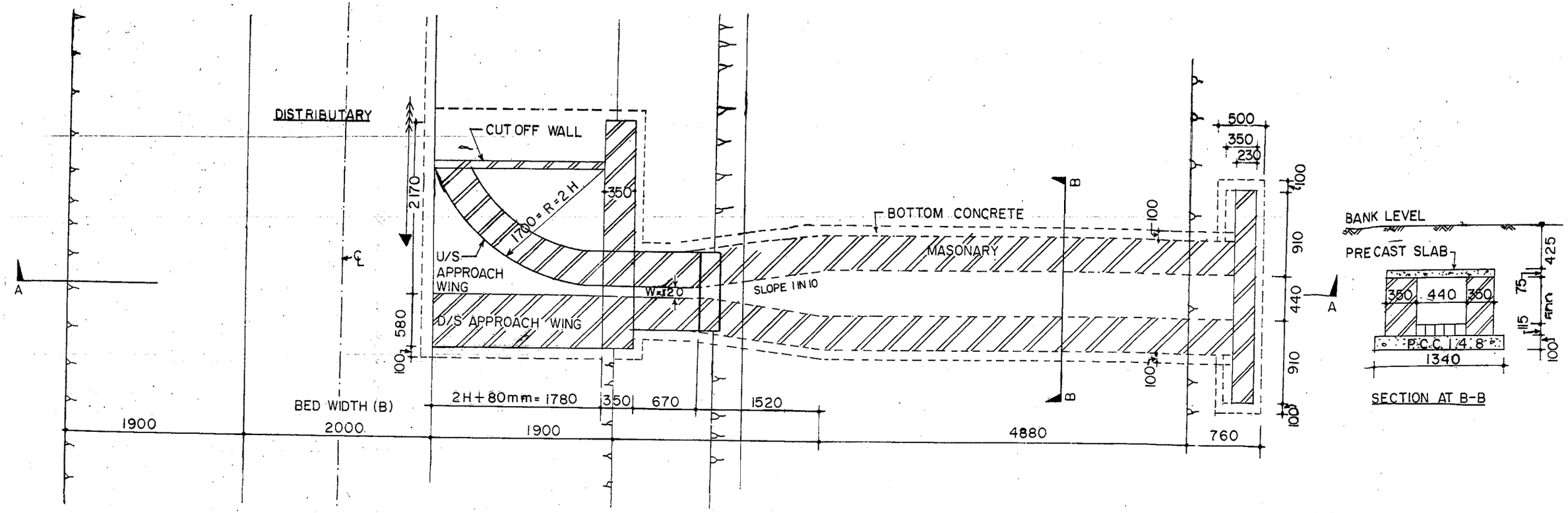
No doubt, in all such circumstances, the supplies shall be reduced or stopped from the *head works,* but the effect of this reduction is felt only after a certain time. Therefore, in order to avoid damage, some immediate action is required, and this is achieved by means of an 'Escape', generally called a **'surplus water escape'.**

The importance of such an escape is realised more, in case, a breach occurs in a channel. If a breach occurs, immediate information is sent to the *head,* so as to close down the channel supplies. But the water already contained in the channel from the head to the breach site will cause wide spread damage if allowed to pass through the breach. In such circumstances, the escape existing on upstream of the breach is opened and most of the water is allowed to escape. *Hence, it can be stated that escapes are the safety valves of canals and must be provided at regular intervals depending upon the importance of the channel and availability of a suitable drainage for the disposal of the escaped water.* The minimum capacity of the escape is generally kept as half of the channel capacity at the point of the escape.

13.6. Types of Canal Escapes

Such an escape may be of the following two types :

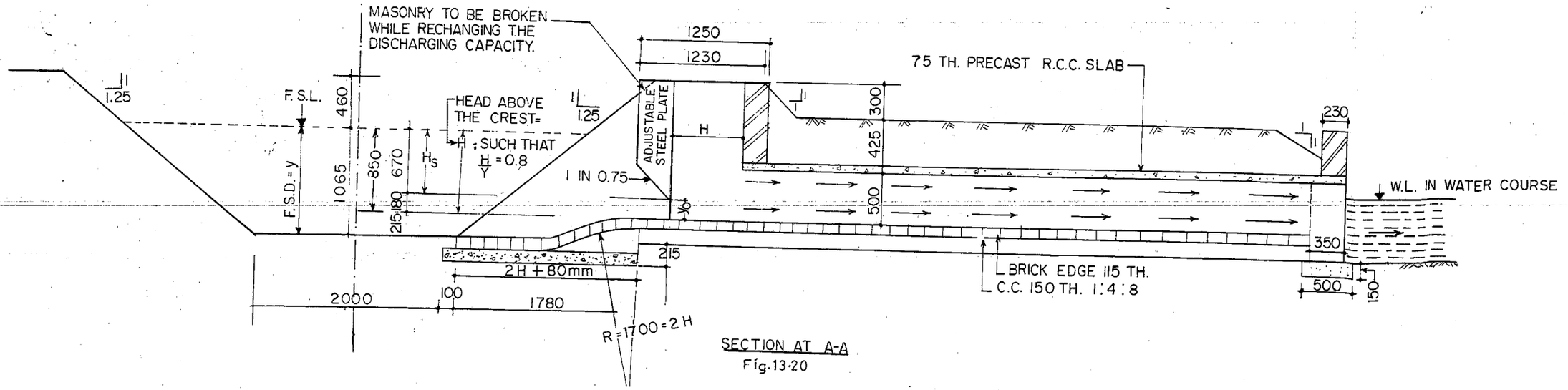
(*i*) **Weir type.** In this type, the crest of the weir wall is kept at R.L. equal to canal FSL, as shown in Fig. 13.9. When water level rises above FSL, it gets escaped.



BOTTOM PLAN OF A.P.M.

ALL DIMENSIONS ARE IN mm.

SCALE = 1:25



SECTION AT A-A

Fig. 13-20

(*ii*) **Regulator type (Sluice type).** In this type, the sill of the escape is kept at canal bed level and the flow is controlled by gates (Fig. 13.10). This type of escapes are preferred these days, as they give better control and can be used for completely emptying the canal.

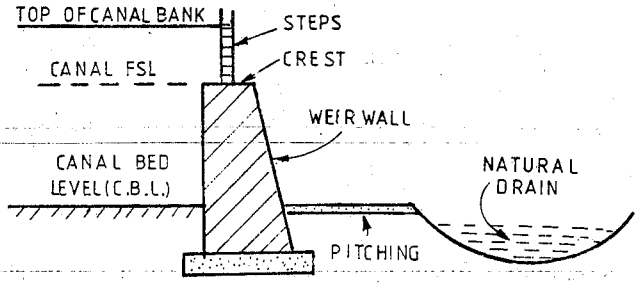


Fig. 13.9. Weir type escape or 'Tail escape'.

The weir type escape may, however, be provided at the tail end of the canal and is useful in maintaining the required FSL in the tail reaches of the canal. This escape is then called a *Tail escape.*

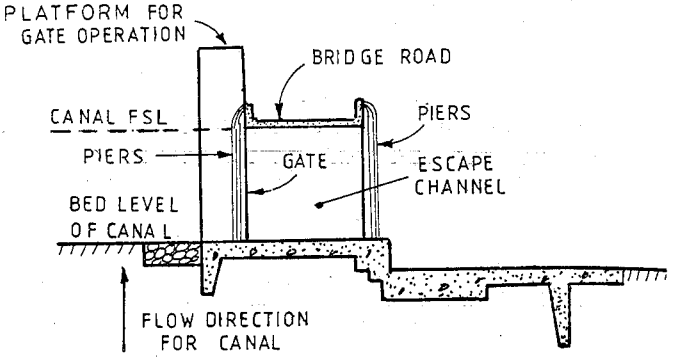


Fig. 13.10. Regulator type escape or 'Surplus water escape'.

The 'regulator type' may, however, be constructed for the purpose of scouring off excess bed silt deposited in the head reaches, from time to time. In that case, it is known as a *Scouring escape.*

In such a case, the sill of the regulator is generally kept at about 0.3 m below the canal bed level at the escape site. When the deposited silt is to be scoured out ; a discharge higher than the full supply discharge is let off into the canal from its *head.* The gates of the escape are raised so as to produce scouring velocities, which scour out the deposited silt. For better efficiency, a cross regulator may be provided just downstream of scouring escape. Such escapes have become obsolete, because *silt ejectors* are now-a-days provided for better efficiency.

The regulator type escape can be designed like a head regulator without any raised crest.

METERING FLUMES

A meter is a structure constructed in a canal for measuring its discharge accurately. A metering flume is an artificially flumed (narrowed) section of the channel, which can be utilized for calculating the discharge in the channel.

The normal upstream section of the channel is narrowed by masonry walls with a splay of 1 : 1 to 2 : 1 to a rectangular section called *Throat.* From where, the channel is slowly diverged so as to attain its normal section by means of masonry wings with a splay of 2 : 1 to 10 : 1. More gradual the convergence and divergence, less will be the loss of head in the flume.

13.7. Types of Metering Flumes

There are two types of metering flumes which are generally used, and work on the principle of Venturimeter. They are :

(*a*) Non-modular venturi flume or drowned venturi-flume, generally called venturi flume.

(*b*) Standing wave flume; or modular venturi flume; or free flow venturi flume.

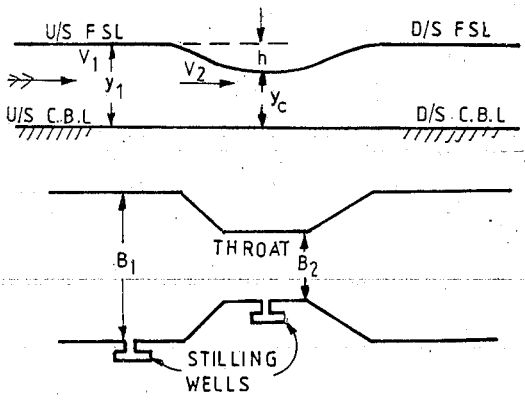


Fig. 13.11. Venturi flume.

(*a*) **Venturi Flume.** A venturi flume consists of a gradually contracting channel leading to 'throat', and a gradually expanding channel leading away from it. Stilling wells are provided for measuring head at the entrance and at the throat, as shown in Fig. 13.11. If h is the difference of head between the two wells, then the discharge is given by

$$Q = C_d \frac{a_1 \cdot a_2}{\sqrt{a_1^2 - a_2^2}} \sqrt{2gh} \qquad ...(13.3)$$

where C_d = Varies from 0.95 to 1.0

a_1 = Area at entrance = $B_1 \cdot y_1$

a_2 = Area at throat = $B_2 \cdot y_2$

When all the factors are known, discharge can be calculated.

(*b*) **Standing Wave Flume.** When a standing wave, (*i.e.* hydraulic jump) forms on the downstream glacis in the diverging channel, the flume is called a standing wave flume (see Fig. 13.12). It is superior to venturi flume because its discharge depends only upon the upstream head over the crest of the throat, and also for the same upstream head, its discharging capacity is more than that of a venturi flume. The length of its throat is atleast 2 to 3 times the head over the crest. Discharge formula for such a flume is given by

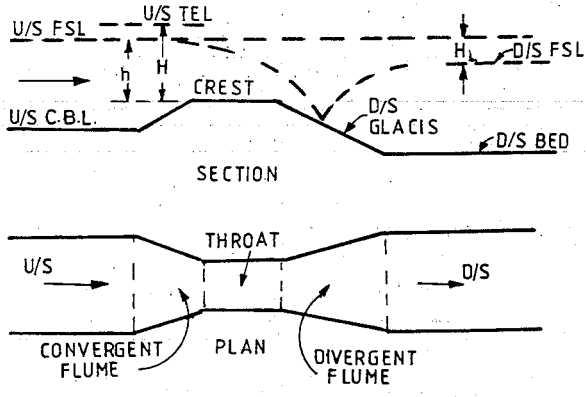


Fig. 13.12. Standing wave flume.

$$Q = 1.7\, C_d \cdot B \cdot H^{3/2} \qquad ...(13.4)$$

where B is the width of the throat.

The head of water over the crest can be measured in a stilling well and the discharge can be easily calculated.

The only drawback of such a flume is that it requires greater loss of head (H_L), and where this loss of head is not available, this flume will act like a venturi flume. Due to this reason, canal fall sites (giving sufficient loss of head, H_L) are generally used as standing wave flumes. For this purpose, flumed meter falls are constructed as explained earlier.

Such a flume is, therefore, seldom constructed separately, and falls, regulators and other masonry structures are flumed, so as to act like a metering flume.

CANAL OUTLETS OR MODULES

A canal outlet or a module is a small structure built at the head of the watercourse so as to connect it with a minor or a distributary channel.

The control and maintenance of the entire network of canals upto the module falls under the jurisdiction of the State Government, and beyond the module, the entire working of the water courses or field channels is taken care of by the cultivators themselves. The module is thus a connecting link between the Government and the cultivators. These outlets play a very prominent role in controlling the flow of water to different areas, so as to effect an equitable distribution of available water in accordance with the needs of the whole area.

Hence, an outlet should be such that it fits well to the decided principles of water distribution.

Moreover, it should be simple, cheaper, efficient and easy to handle. The design of the outlet is, therefore, of prime importance in efficient and effective management of canal irrigation system. The requirements of a good module are listed below :

13.8. Requirements of a Good Module

(1) The module should fit well to the decided principles of water distribution. For example, if the supply is to be fixed in accordance with the cultivable area commanded by the outlet, the outlet must be able to pass a constant and a fixed discharge. Similarly, if the supply is to be regulated in accordance with the area irrigated in the past year, the capacity of the outlet should be capable of being changed from year to year.

(2) The module should be simple, so that it can be easily constructed or fabricated by local masons or technicians.

(3) It should work efficiently with a small working head.

(4) The outlet should be cheaper, since they are required in large numbers.

(5) It should be sufficiently strong with no moving parts, thus avoiding periodic maintenance.

(6) The outlet should be such as to avoid interference by cultivators, thus preventing undue tapping of water by cultivators. In other words, any interference by the local people should be made difficult and easily detectable.

(7) It should draw its fair share of silt.

13.9. Types of Outlets (Modules)

The various available types of outlets can be classified into three classes :

(*i*) **Non-modular outlets** are those through which the discharge depends upon the difference of head between the distributary and the water-course. The discharge through such a module, therefore, varies widely with either a change in the water level of the distributary or that of the water-course. The common examples of this type of outlets are : (*i*) *open sluice,* and (*ii*) *drowned pipe outlet.*

(*ii*) **Semi-modules or Flexible modules** are those through which the discharge is independent of the water level of the water-course but depends only upon the water level of the distributary so long as a minimum working head is available. The discharge through such an outlet will, therefore, increase with a rise in the distributary water surface level and *vice versa.* The common examples of this type of modules are : *pipe outlet, venturi flume, open flume and orifice semi-module.*

(*iii*) **Rigid modules or Modular outlets** are those through which the discharge is constant and fixed within limits, irrespective of the fluctuations of the water levels of either the distributary or of the water course or of both. *Gibb's module* is a common example of such a module.

13.10. Criteria for Judging the Performance of Modules

The behaviour and functioning of a module can be judged by the following important terms and definitions :

Flexibility. Flexibility is defined as the ratio of the rate of change of discharge of the outlet to the rate of change of discharge of the distributary channel.

Thus $$F = \left[\frac{dq/q}{dQ/Q}\right] \quad ...(13.5)$$

where F = Flexibility of the outlet

q = Discharge passing through the outlet

Q = Discharge in the distributary channel.

Now, if H is the head acting on the outlet, the discharge through the outlet (q) may be expressed as

$$q = C \cdot H^m \quad ...(13.6)$$

where C and m are constants depending upon the type of outlet.

Similarly, the discharge passing down the distributary channel may be expressed as

$$Q = K \cdot y^n \quad ...(13.7)$$

where K and n are constants and y is the depth of water in the distributary.

Differentiating equations (13.6) and (13.7), we get

$$dq = C \cdot m \cdot H^{m-1}\, dH \quad ...(13.8)$$

$$dQ = K \cdot n \cdot y^{n-1}\, dy \quad ...(13.9)$$

Dividing equation (13.8) with (13.6), we get

$$\frac{dq}{q} = \frac{C \cdot m \cdot H^{m-1}\, dH}{C \cdot H^m} = \frac{m}{H} dH \quad ...(13.10)$$

Similarly,

$$\frac{dQ}{Q} = \frac{K \cdot n \cdot y^{n-1}\, dy}{K \cdot y^n} = \frac{n}{y} dy \quad ...(13.11)$$

Hence, the flexibility,

$$F = \frac{\frac{dq}{q}}{\frac{dQ}{Q}} = \frac{\frac{m}{H} dH}{\frac{n}{y} dy} = \frac{m}{H} \cdot \frac{y}{n} \cdot \frac{dH}{dy} = \frac{m}{n} \cdot \frac{y}{H} \frac{dH}{dy}$$

or $$F = \frac{m}{n} \cdot \frac{y}{H} \cdot \frac{dH}{dy} \quad ...(13.12)$$

But since a change in the water depth of the distributary (dy) would result in an equal change in the head working on the outlet (dH), we have

$$dy = dH.$$

Substituting in equation (13.12), we get

$$\boxed{\text{Flexibility} = F = \frac{m}{n} \cdot \frac{y}{H}} \quad ...(13.13)$$

Proportionality. The outlet is said to be proportional when the rate of change of outlet discharge equals the rate of change of channel discharge. In other words, the outlet is 'proportional' when 'flexibility' equals unity. Hence for a proportional outlet, we get

$$F = \frac{m}{n} \cdot \frac{y}{H} = 1$$

or
$$\frac{H}{y} = \frac{m}{n} = \frac{\text{outlet index}}{\text{channel index}} \qquad ...(13.14)$$

Setting. The ratio H/y, *i.e.* the ratio of the depth of the sill level of the outlet below the FSL of the distributary, to the full supply depth of the distributary, is known as *setting*. Thus, for a proportional outlet

H/y, *i.e.* 'outlet setting' should be made.

$$= \frac{\text{outlet index}}{\text{channel index}}.$$

For a wide trapezoidal channel, the discharge is proportional to $y^{5/3}$. Therefore, for such channels

$$n = 5/3$$

Hence, the channel index is generally 5/3. The discharge through an orifice type outlet is proportional to $\sqrt{H}$; and thus for such an outlet, index $m = \frac{1}{2}$.

$$\therefore \text{ Setting } = \frac{H}{y} = \frac{m}{n} = \frac{1/2}{5/3} = 0.3$$

Hence, for such a combination of an orifice type outlet and a trapezoidal channel, the setting must be equal to 0.3. In other words, a pipe or orifice type outlet shall be proportional, if the outlet is fixed or set at 0.3 times the depth below the water surface.

Similarly, for a weir type outlet, $m = 3/2$ ($\because$ the discharge is proportional to $H^{3/2}$).
Hence, the setting for a combination of a weir type outlet and a trapezoidal channel

$$= \frac{m}{n} = \frac{3/2}{5/3} = 0.9$$

Hence, for the weir type outlet to be proportional, the outlet should be set at 0.9 times the depth below the water surface.

Hyper-proportional outlet. An outlet is known to be **hyper-proportional** if its flexibility is greater than unity.

Thus $F = \frac{m}{n} \cdot \frac{y}{H} > 1$ or $\frac{m}{n} > \frac{H}{y}$

or $\frac{H}{y} < \frac{m}{n}$ or $\text{Setting} < \frac{m}{n}$

Hence, the outlet will be hyper-proportional, if the numerical value of its setting (H/y) is less than m/n. In other words, the outlet is hyper-proportional if set higher (*i.e.* head acting on the outlet, H, is less than what is required for proportionality).

Sub-proportional outlet. An outlet is known to be **sub-proportional** if its flexibility is less than unity. Thus

$$F = \frac{m}{n} \frac{y}{H} < 1 \quad \text{or} \quad \frac{m}{n} < \frac{H}{y} \quad \text{or} \quad \frac{H}{y} > \frac{m}{n}.$$

Hence, the outlet will be sub-proportional, if the numerical value of its setting (H/y) is more than m/n. In other words, the outlet is sub-proportional if set lower (*i.e.* H is more than what is required for proportionality).

Sensitivity. Sensitivity is defined as the ratio of the rate of change of discharge through the outlet to the rate of change of water level of the distributary, referred to the normal depth of the channel. For rigid modules, the discharge is fixed and hence the sensitivity is zero.

For flexible modules, where the discharge through the outlet is independent of the water level of the watercourse and depends only upon the water level of the distributary, a gauge can be fixed and calibrated so as to indicate its reading $G = 0$ when $q = 0$. Thus

$$\text{Sensitivity} = S = \left[\frac{dq/q}{dG/y}\right] \qquad ..(13.15)$$

Relation between Sensitivity and Flexibility. We know that

$$\text{Flexibility} = F = \left[\frac{dq/q}{dQ/Q}\right] \quad \textit{i.e.} \text{ Eq. (13.5)}$$

But $\frac{dQ}{Q} = \frac{n}{y}\,dy$ *i.e.* Eq. (13.11)

Equation (13.5) then becomes

$$F = \left[\frac{dq/q}{(n/y)\,dy}\right] = \frac{1}{n}\left[\frac{dq/q}{dy/y}\right]$$

But since, $dG = dy$ we get $F = \frac{1}{n} \cdot S$.

or $$\boxed{S = n \cdot F} \qquad ...(13.16)$$

If $n = \frac{5}{3}$, for wide trapezoidal channels, then, $S = (5/3)\,F$.

It is also evident that :

(*i*) Greater is the variation in the discharge of an outlet for a given rise or fall in water surface level of the distributary, the larger is the sensitivity of the outlet.

(*ii*) The sensitivity of a non-flexible module is zero.

13.11. Certain other Important Definitions Connected with Modules

Minimum modular head. The minimum difference between the upstream and downstream water levels, which is required to be maintained so as to enable the module to pass the designed discharge is known as *minimum modular head* or *minimum modular loss.*

Efficiency of an outlet. It may be defined as the ratio of the head recovered to the head put in. Lesser is the head required for functioning of the outlet ; more efficient the outlet will be. Efficiency is a measure of the conservation of head by the outlet.

Drowning Ratio. It is the ratio of the depth of water level over crest on the downstream of the module to the depth of water level over crest on the upstream of module. In case of a weir type outlet, the efficiency is the same as the drowning ratio.

Modular limit and modular range. The modular limits are the extreme values of any one or more variables, beyond which an outlet becomes incapable of acting as a module or semi-module. The range between the lowest and the highest limiting values of various such factors is known as *modular range.*

13.12. Types of non-Modular Outlets

A non-modular outlet may be in the form of a rectangular opening or *open sluice,* or a *simple submerged pipe.* Pipe outlet (when submerged) is a very simple type of a non-modular outlet and is extensively used in South India.

13.12.1. Open Sluice. An open sluice (Fig. 13.13), like a bridge opening, is a rectangular pucca opening created across the bank of the distributary, by raising two abutments at 2.5 to 3 m apart and with a horizontal pucca floor. The width B of the opening and height H of the opening are computed to pass the given discharge, by using the appropriate discharge formula.

The sill level of the outlet (*i.e.* the sill level of the pucca floor) is kept somewhat above the DBL of the distributary, as shown in Fig. 13.13. For maximum silt draw, however, the height of water level in distributary above sill level, *i.e*, H, has to be kept equal to FSD of distributary, y.

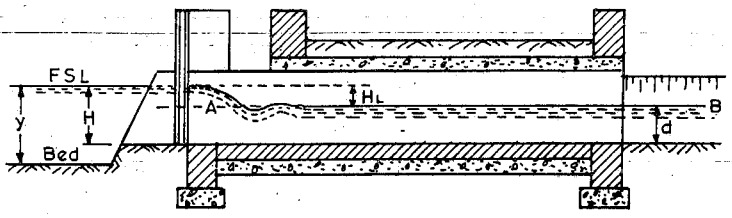


Fig. 13.13. Open Masonry Sluice.

There was a time when the sectional area of the opening (B.H) was calculated by assuming a flow velocity of 0.3 m/sec through the opening, to pass the given discharge, q [*i.e.* Area $(BH) = q/0.3$] through the outlet. The old simple calculation method is even being used in present days for sluices off-taking from the river. But as owners of the water-courses could draw extra discharge in a simple open cut by lowering & clearing their water courses, such open cuts are nowadays designed on the basis of drowned weir formula by computing the average head loss (H_L). H_L is given as = [the water level of distributary-water level of water course]. H_L in design is, thus, taken as equal to the average of head loss values observed once a fortnight throughout the season.

The discharge through the opening is, thus, given by the *supressed weir* (*i.e.* weir without end contractions)* *formula, for submerged flow*, where discharge consists of :

(*i*) flow over upper part of section (lying above the line *AB*) of depth H_L (*i.e.* $H - d$) may be considered as free discharge into the air as a weir flow, given by eqn.

$$q_1 = \frac{2}{3} C_{d_1} \cdot \sqrt{2g} \cdot B \cdot H_L^{3/2} \qquad ...(i)$$

(*ii*) flow through the remaining lower part (lying below the line *AB*), which is computed as discharge through a submerged orifice, given as :

$$q_2 = C_{d_2} \cdot B \cdot d \sqrt{2g\, H_L}$$

$$= C_{d_2} \cdot B \cdot (H - H_L) \sqrt{2g \cdot H_L} \qquad ...(ii)$$

If we assume $C_{d_1} = C_{d_2} = C_d$, then the total discharge through the outlet is given as :

$$q = q_1 + q_2 = (i) + (ii)$$

$$= \frac{2}{3} C_d \cdot \sqrt{2g} \cdot B \cdot H_L^{3/2} + C_d \cdot B\,(H - H_L) \cdot \sqrt{2g \cdot H_L}$$

$$= C_d \cdot \sqrt{2g} \cdot B \sqrt{H_L} \cdot \left[\frac{2}{3} \cdot H_L + (H - H_L) \right]$$

$$= C_d \cdot \sqrt{2g \cdot H_L} \cdot B \left[H - \frac{1}{3} H_L \right] \qquad ...(13.17)$$

Taking $C = 0.64$, we get

$$q = 2.83513 \sqrt{H_L} \left[H - \frac{1}{3} H_L \right] \qquad ...(13.18)$$

* Weir having length equal to width of canal, across which the weir is constructed.

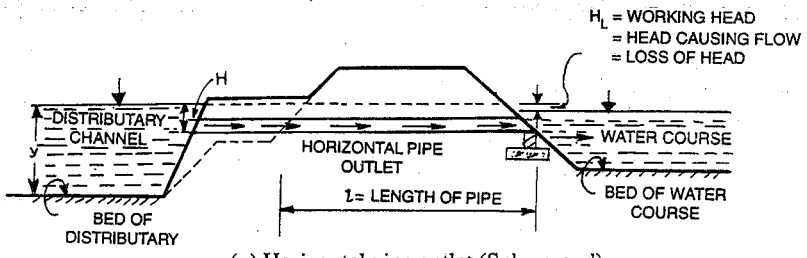
Flexibility of an open sluice is given as

$$= F = \frac{m}{n} \times \frac{y}{H} = \frac{1}{5/3} \times \frac{y}{H} = \frac{3}{5} \frac{y}{H} \qquad ...(13.19)$$

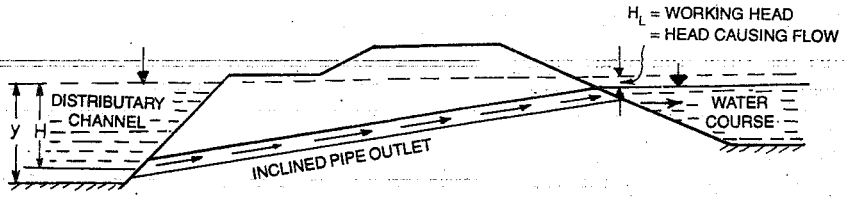
and setting H/y for proportionality $= m/n = 3/5 = 0.6$.

Hence for an open sluice to be proportional, the pucca floor should be provided in such a way as to give $H = 0.6y$; or the sill level of floor be provided at 0.4 y above the DBL of canal.

13.12.2. Submerged Pipe Outlet. Two types of pipe outlets are shown in Figs. 13.14 (a) and (b). The pipe diameter varies from 10 to 30 cm. Pipes are generally embedded in concrete and are generally fixed horizontally at right angles to the direction of flow. They may also be laid sloping upwards by depressing the upstream end of the pipe [as shown in Fig. 13.14 (b)] so as to increase silt conductivity. In U.P., the pipes are generally laid horizontally at about 21 cm below the water surface level of the distributary channel.



(a) Horizontal pipe outlet (Submerged)



(b) Inclined pipe outlet (Submerged)

Fig. 13.14. Non-modular Outlets.

The velocity through the pipe can be precisely computed by using the relation :

H_L = Total loss of head = Entry loss + Frictional loss + Velocity head at exit

or $$H_L = 0.5 \frac{V^2}{2g} + \frac{f' l V^2}{2gd} + \frac{V^2}{2g} \qquad ...(13.20)$$

or $$H_L = \frac{V^2}{2g}\left[1.5 + \frac{f' l}{d}\right] \qquad ...(13.21)$$

where H_L is the difference in the water level of the distributary and the water course, l is the length of the pipe, d is the diameter of the pipe and f' is the coefficient of friction of the material of the pipe.

After computing V from Eq. (13.21), the discharge q can be computed as $q = V \times$ area of outlet pipe.

The discharge, however, for all practical purposes, may be easily computed by using the simple relation :

$$q = C_d \cdot A \cdot \sqrt{2gH_L} \qquad ...(13.22)$$

where q = Discharge through the outlet

C_d = Coefficient of discharge for the pipe outlet. Its average value, determined at CWPRS poona is 0.73, although it depends upon the length & size of the pipe, besides on the pipe material.

A = Area of the pipe

H_L = Difference of head between the FSL of distributary and FSL of water course, usually called working head of irrigation outlet.

It becomes evident from equation (13.22), that the discharge through the outlet can be increased by increasing H_L or by lowering the bed of the watercourse. Hence, for a certain water level in the distributary, the water entering high fields shall be less and that entering the lower fields shall be more ; consequently equitable distribution of water becomes difficult, as the owners of high fields are put to a disadvantage in drawal of their share.

Although, such outlets can work even with very small available heads, and silt conduction is also quite good due to disturbance at the entrance, they may not be preferred as they do not ensure equitable distribution of water.

Pipe outlets are more in vogue in South India, where glazed earthenware pipe lengths, 0.6m long, are used to build up the barrel. Against the usual convention of sockets facing against the flow, the pipes here are usually laid in the reverse direction, otherwise the head (front) socket will become a bellmouth at entry of discharge from the distributary, making the measurement of diameter difficult and unreliable. The spigot of the pipes are hence kept at the upstream ends.



Fig. 13.14(*c*). Pipe outlet controlled by Shutter on u/s end.

A non-modular outlet may be controlled by providing a shutter on its upstream end (see photographic view, Fig. 13.14(*c*)). Arrangement is provided to lock the shutter in any required position, so as to have a given discharge pass through the outlet (*i.e.* area of opening A can be changed and the discharge passing is controlled). *Since the loss of head in a non-modular outlet is less than that in a modular outlet, the former is useful where much loss of head is not available.*

Flexible modules (semi modules) and rigid modules are preferred to non-modular outlets, so as to ensure more equitable distribution of water in the watercourses, irrespective of their being at high or low levels. However, such outlets can work as modular outlets, only within certain limits of water level in the distributary and the water course. The range over which each module works as modular outlet, is called its *working range;* or *range of modularity.* Also certain minimum difference of water levels on two sides of each module should always be there to ensure its modularity (*i.e.* its working as modular outlet). This minimum difference of levels is called *minimum modular head* or minimum *loss* of head of the modular outlet.

Example 13.1. *Design an irrigation outlet for the following data :*

FSQ of outlet	*= 50 lit/sec.*
FSL in distibutary on u/s side of outlet	*= 200.00 m.*
FSL in water course on d/s side of outlet	*= 199.92 m.*
FSD in distributary on u/s side of outlet	*= 1.05 m.*

Solution.

Available head across the outlet

= FSL of Distributary – FSL of water course

= 200.00 – 199.92 = 0.08 m.

Since the available head is very small, a non-modular outlet (such as a submerged pipe outlet) will have to be provided [∵ The very small head of 0.08 m clearly indicates that the d/s water level will definitely be above the opening on that side.]

The discharge in such a submerged pipe outlet is given by Eq. (13.22) as :

$$Q = C_d \cdot A \cdot \sqrt{2g\, H_L}$$

where Q = 50 lit/sec = 0.05 cumec.

A = Area of pipe

H_L = working head or Loss of head between u/s and d/s.

= 0.08 m

C_d = 0.73 for submerged flow.

Putting these values in the above Eqn. we get

$$0.05 = 0.73\, A \cdot \sqrt{2 \times 9.81 \times 0.08} \quad \text{or} \quad A = 0.055\ \text{m}^2$$

If a pipe of dia d is used, then

$$\frac{\pi}{4} d^2 = 0.055\ \text{m}^2 \quad \text{or} \quad d = 0.264\ \text{m}.$$

Hence, use a pipe of dia, say 30 cm.

The R.L. of the bed of the distributary

= 200.00 – 1.05 = 198.95 m.

The pipe top can be fixed at about 22 cm below the FSL of the distributary. In other words, the pipe can be laid horizontally with its invert level (or sill level)

= 200.00 – 0.22 – 0.30 = 199.48 m

i.e. at 199.48 – 198.95 = 0.53 m above the bed of the distributary.

13.13. Types of Semi-Modules or Flexible Outlets

The common types of semi-modules are :

(*i*) *Pipe outlet discharging freely into the air.*

(*ii*) *Venturi-flume outlet or Kennedy's Gauge outlet.*

(*iii*) *Open flume outlet.*

(iv) Adjustable orifice semi-module.

These outlets are discussed below in details :

13.13.1. Free Pipe Outlet. Pipe outlet discharging freely into the atmosphere is the simplest and the oldest type of a flexible outlet. The discharge through such an outlet will depend only upon the water level of the distributary, and will be independent of the water level of the water-course so long as the pipe is discharging freely. Silt conduction for such an outlet is quite good and efficiency is high. But a freely falling jet outlet can be provided only at a few places where sufficient level difference between the distributary and water-course is available. The discharge can be easily computed by using the equation.

$$Q = C_d \cdot A \cdot \sqrt{2g H_0} \qquad ...(13.23)$$

where C_d is coefficient of discharge = 0.62 for average condition of free over fall.

H_0 = Head on u/s side measured from FSL of distributary up to the centre of pipe outlet.

A = Area of cross-section of pipe

Example 13.2. *Design a pipe outlet for the following data :*

Full supply discharge at the head of water course = 90 lit/sec

FSL in distributary = 205.00 m.

FSL in water course = 204.00 m.

Solution

Available head across the outlet

= FSL of distributary – FSL of water course

= 205.00 – 204.00 = 1 m.

This available head of 1 m is sufficient enough to make the pipe outlet discharge freely into the water course, as the d/s end of the pipe can be fixed below the water level of the water course, thus making it a semi-module.

The discharge through such an outlet is given as :

$$Q = C_d \cdot A \sqrt{2g H_0}$$

where C_d = 0.62 for free pipe outlet.

H_0 = Head on upstream side above the centre line of pipe.

Q = 9.0 lit/see = 0.09 cumec.

Assuming the dia of the pipe as 25 cm, we have

$$0.09 = 0.62\left[\frac{\pi}{4} \times (0.25)^2\right]\sqrt{2 \times 9.81 H_o}$$

$$= 0.62 \times 0.049 \times 4.43 \sqrt{H}$$

$$\sqrt{H_o} = 0.657 \text{ m.}$$

$$\therefore \quad H_o = 0.44 \text{ m}$$

∴ R.L. of the centre of outlet pipe

= 205.00 – 0.44 = 204.56 m.

R.L. of invert of outlet pipe (*i.e.* sill level)

$$= 204.56 - \frac{0.25}{2}$$

= 204.43 m > FSL of water course *i.e.* 204.00 m.

Hence, a pipe of 25 cm dia can be laid horizontally with its bottom or sill level at RL 204.43 m, and it will be discharging freely as a semi-module. **Ans.**

13.13.2. Venturi Flume Outlet or Kennedy's Gauge Outlet. Kennedy's Gauge Outlet is of a Venturi flume type and is shown in Fig. 13.15. It is made of cast iron and consists of three main parts :

(*a*) an orifice with a bell mouth entry ;

(*b*) a long expanding delivery pipe ;

(*c*) an air-vent connecting the throat of the delivery pipe to the atmosphere.

Water from the distributary enters this module through the bell-mouthed entry. The diameter is narrowest at the throat and the water jet shoots across the air in the base of the vertical air bent pipe. The base of the vent pipe is kept conical and air holes are provided at the top, so as to allow free circulation of air around the jet. In other words, the jet is equivalent to be discharging freely into the atmosphere, thereby making the discharge independent of the water level in the water-course. The jet, then passes through the cast iron expanding pipe (about 3 metres long) and then through its concrete extension and finally discharges into the watercourse, as shown in Fig. 13.15.

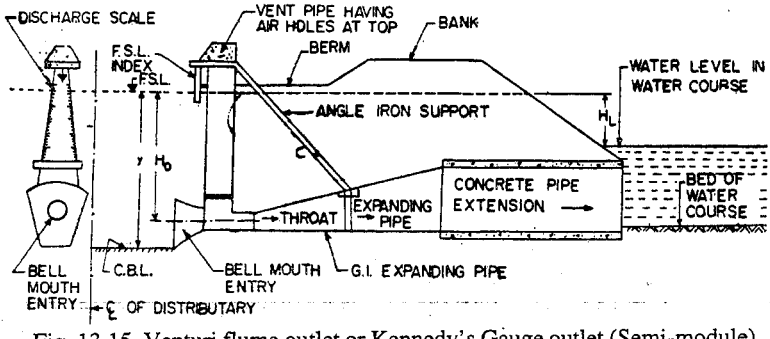


Fig. 13.15. Venturi flume outlet or Kennedy's Gauge outlet (Semi-module).

The discharge through this type of outlet is given as :

$$q = C_d \cdot A \sqrt{2g H_0} \qquad ...(13.24)$$

where q = Discharge through the outlet

A = Area of cross-section of the pipe at the throat.

H_0 = Head measured from FSL of distributary to the centre of the pipe at throat.

C_d = Coefficient of discharge which may be as high as 0.97.

Kennedy's gauge outlet is an old one and was designed in 1906, by R.G. Kennedy, Chief Engineer, Punjab Irrigation Department. This type of a module, though having the properties of semi-modularity, was very easy to tamper with. The farmers used to block the air holes so as to increase the outlet discharge. When the air bent is closed, the jet sucks the surrounding air, and pressure falls below the atmosphere, which increases the discharge. Hence, due to its tampering defect and due to its high cost of construction, *this type of module has almost become obsolete these days.*

13.13.3. Open Flume Outlet. It is a weir type outlet with a constricted throat and an expanding flume on the downstream, as shown in Fig. 13.16. Due to the constriction,

a super-critical velocity is ensured in the throat and thereby allowing the formation of a jump in the expanding flume. The formation of hydraulic jump makes the outlet discharge independent of the water level in the water-course ; thus making it a semi-module. There are many types of such an outlet.

The most commonly used type of such an outlet is called the Punjab Open Flume Outlet, as the same was first used in Punjab. A typical plan and section of such a module are shown in Fig. 13.16. A photographic view of a distributary installing such modules is also shown in Fig. 13.17.

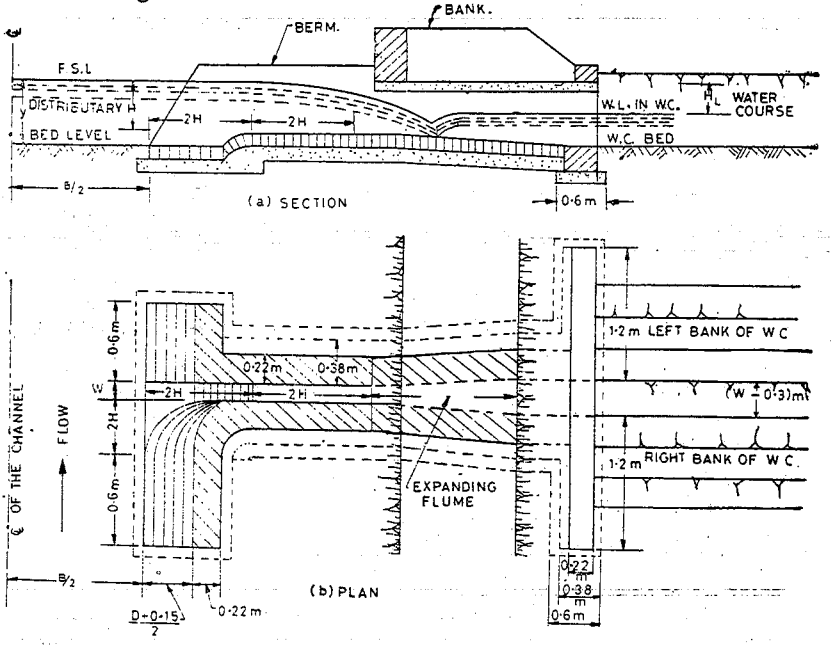


Fig. 13.16. Punjab Open Flume Outlet (Semi-module).

In such a Punjab open flume outlet, the upstream approach is made curved and the downstream approach is made straight, as shown in Fig. 13.16 (*b*). The length of the throat is kept $2H$. The width of the flume at the downstream end is kept 0.3 m wider than the throat width (W), as shown. The slope of the d/s glacis is kept to the water course.

The discharge through such an outlet is given as :

$$q = C_d \cdot WH^{3/2} \qquad ...(13.25)$$

where q = Discharge through the outlet

H = Head over the crest measured from FSL of distributary

W = Width of the throat $\approx \frac{q}{Q} A$, where

Q = discharge of distributary and

A = Area of waterway of distributary of bed width B and depth y

C_d = Coefficient of discharge, having a theoretical value equal to 1.71 in M.K.S. or S.I. units.

Due to the various losses, the actual value of C_d is less than 1.71, and depends upon the width of the throat, as shown in Table 13.1.

Table 13.1

Throat width in metres (W)	*Value of C_d*
0.06 to 0.089	1.60
0.09 to 0.119	1.64
Over 0.12	1.66



Fig. 13.17. Photoview of the tail end of a minor distributary in W.J.C. System in Haryana. Three water courses are taking off from this minor, and the three outlets provided are of open flume type.

For such a module to be proportional, setting is given by

$$\frac{H}{y}=\frac{m}{n}=\frac{3/2}{5/3}=0.9 \qquad ...(13.26)$$

Its efficiency varies between 80 to 90%. The minimum modular loss involved in such an outlet is about 0.2 *H*.

13.13.4. Adjustable Orifice Semi-Modules. Various types of orifice semi-modules have been designed since olden days. The one which found a lot of popularity is called **Crump's adjustable proportionate module (APM)** (see Figs. 13.18 and 13.19). Further improvements in approaches etc. have since been carried out in crump's APM, and the latest model, which is now used in Punjab and Haryana, is called an Adjustable orifice semi-module (A.O.S.M.). Typical dimensions of such an outlet are shown in the attached chart Fig. 13.20.

This type of an adjustable module is considered to be the best of all the modules and is mostly adopted. An adjustable orifice module consists of an orifice provided with a gradually expanding flume on the downstream side of the orifice. The flow through

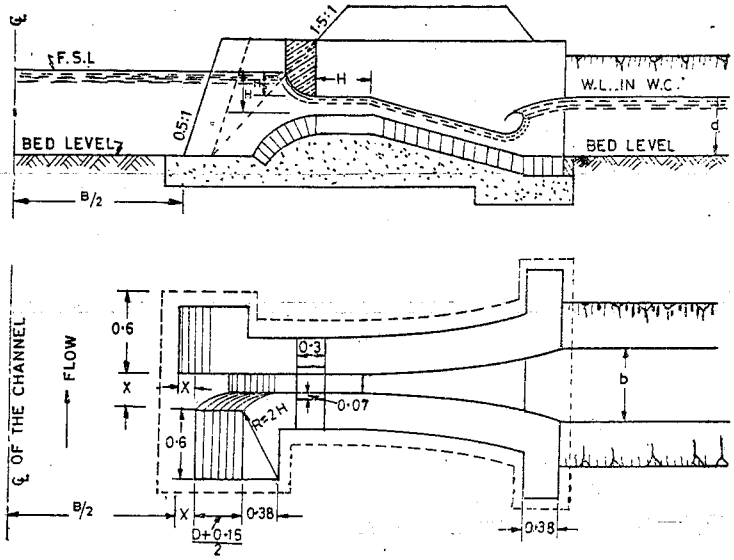
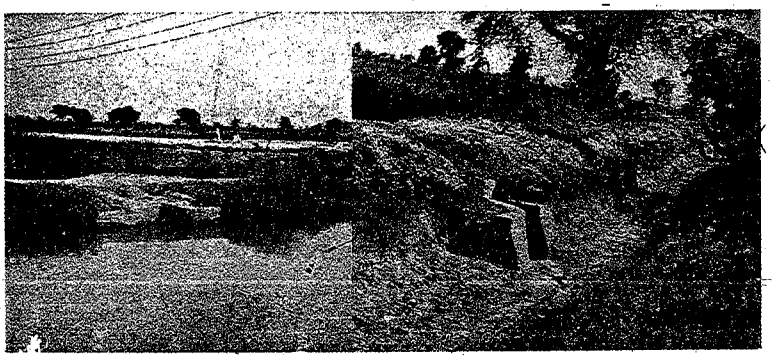


Fig. 13.18. Crump's A.P.M. outlet (Orifice-Semi-module)



(*a*) (*b*)

Fig. 13.19. Photoview of two A.P.M. Outlets. The common type of outlet used in Northern India. The distributary in Fig. 13.19 (*b*) is flowing full, and that in Fig. 13.19 (*a*) is dry. The size of the outlet is fixed on the basis of CCA. There is no control over the discharge. The water flows through the outlet so long as the canal water surface is above the bottom of the outlet opening.

the orifice is super-critical, resulting in the formation of a hydraulic jump in the expanding flume position. The formation of jump makes the discharge independent of the water level in the water-course.

The principal features of an adjustable orifice module are similar to those of a flumed regulator with horizontal crest and curved water approach on the upstream, and downstream wings expanding to the width of the water course, b. But unlike gates, it is provided with cast iron roof block, around which masonry is done. The opening height, y_0, can be changed by suitably adjusting the roof block, which can be easily done after dismantling the masonry around it. Since the roof block cannot be readjusted without breaking the masonry around it, the opening, y_0, and hence the outlet discharge, cannot be easily tempered with by the cultivators. The module is thus perfectly rigid, and at the same time adjustable in dimensions at a slight cost of re-doing the masonry.

The discharge through such an outlet is given by the formula :

$$q = C_d \cdot (W \cdot y_0) \cdot \sqrt{2g\, H_s} \qquad ...(13.27)$$

where q = Discharge through the outlet

W = Width of the throat

y_0 = Height of the orifice opening (smallest)

= generally kept 1.5 to 2 times the width (W) of the opening.

H_s = Head measured from the upstream water level in the distributary to the lowest point of the roof block.

C_d = Coefficient of discharge, whose value varies between 0.8 to 1.05 for throat width (W) varying between 0.06 to 0.3 m. It may be safely taken for normal throat width (of the order of 0.12 m or so) as 0.91, in which case the above formula reduces to

$$q = 4.04 \cdot W \cdot y_0 \sqrt{H_s} \qquad ...(13.28)$$

This type of adjustable modules are provided in eight different standard widths = 0.06, 0.075, 0.10, 0.12, 0.15, 0.19, 0.24 and 0.3 metres.

The *minimum modular head* loss involved in such a module is given by the formula

$$H_L = 0.82\, H_s - \frac{W}{2} \qquad ...(13.29)$$

Originally, when this module had a setting (*i.e.* H/y) of 6/10, it aimed at exact proportionality and, therefore, used to be called A.P.M. (Adjustable Proportional Module). But experience showed that the channels using such modules silted very badly, and hence proportionality was sacrificed to enable the outlet to carry higher silt charge by increasing the setting to 8/10. Since then, the outlet is known at A.O.S.M. (Adjustable Orifice Semi-module).

A few rules which are of use in fixing the dimensions of the latest model of AOSM have been indicated in the Fig. 13.20 itself.

Advantages of this type of outlet are :

(*i*) The adjustment can be made by raising or lowering the roof blocks at low costs by dismantling the masonry in which the roof bolts are fixed.

(*ii*) Any undue tampering of roof blocks by the cultivators can be easily detected, as it requires the breaking of the visible masonry key.

(*iii*) It is simple and cheaper.

(*iv*) Silt conduction is good at various settings, such as shown below

Setting	*Silt Conduction*
$\frac{6}{10}$ th Setting	99.5%
$\frac{8}{10}$ th Setting	109.7%
$\frac{10}{10}$ th Setting	121.9%

13.14. Types of Rigid Modules

There are a few types of rigid modules which have no moving parts, such as :

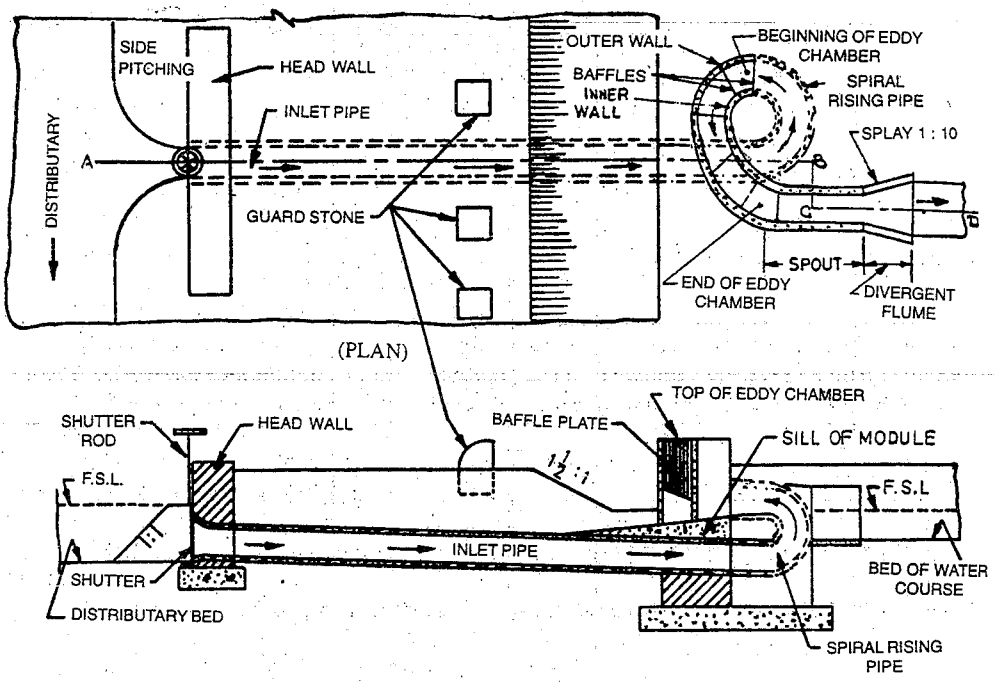
(*i*) *Gibb's module ;*

(*ii*) *Khanna's rigid module ;* and

(*iii*) *Foote module.*

Out of all these modules, Gibb's module is the most important and widely used. It is described below :

13.14.1. Gibb's Rigid Module. The plan and section of Gibb's module are shown in Fig. 13.21. Gibb's module was designed by A.S. Gibb, formely Executive Engineer in Punjab Irrigation Deptt. The discharge range of Gibb's module varies between 0.03 to 0.45 cumec (1 to 16 cusecs). For lower values of discharge, *i.e.* between 1 to 3 cusecs, it is often economical to construct it in R.C.C., but for higher discharges (greater than 3 cusecs) it may be constructed in brick masonry.



(Section A-B-C-D)

Fig. 13.21. Gibb's Module.

Gibb's module consists of an inlet pipe having a bell mouth. Water entering this inlet pipe is directed to the eddy chamber through a 180° rising spiral pipe in which free vortex flow is developed. The upstream end of the inlet pipe is controlled by a shutter. The rectangular eddy chamber having a horizontal floor is generally semi-circular in plan (although it may be a circle or $1\frac{1}{2}$ circle depending upon discharge and range of working required for the module). The eddy chamber is fitted with a number of baffle plates (minimum six numbers) placed equidistant along its circular path. The lower edge of each baffle, slopes up from the inner wall to the outer wall of the eddy chamber.

The free vortex flow generated in the eddy chamber ensures, that, velocity × radius = constant, for all filaments. Thus, the water at the outer wall of the chamber having greater radius will have lesser velocity, resulting in a rise of water level there. Hence, the water surface level in the eddy chamber will slope down from the outer wall towards the inner wall. The baffle plates inserted in the eddy chamber, sloping up towards the outer wall, at required heights above the sill of the module, will help in preventing any excess amount of discharge likely to pass through the module. If the head causing flow increases, water banks up at the outer wall and impinges against the baffles, imparting an upward rotational flow to the water, which spins round in the compartment between two successive baffles, and finally drops on the oncoming stream of water ; thus dissipating excess energy. This helps in keeping the discharge constant for a wide range of variations in the head. The number of baffles coming into the action depends upon the variation in the head, as more and more baffles come into action with more and more discharge. The device thus maintains a constant discharge.

The discharge through the Gibb's module is given by Gibb's formula as :

$$q = r_2 \cdot \sqrt{2g}\left(y_1 + h_2\right)^{1.5}\left[\frac{m^2-1}{m^3}\log_e m + \frac{1}{m}\log_e m - \frac{m^2-1}{2m^3}\right] \quad ...(13.30)$$

where r_1 = radius of inner semi-circle

r_2 = radius of outer semi-circle

$m = \dfrac{r_1}{r_2}$

y_1 = Depth of water at inner circumference

h_2 = Head at outer circumference.

Gibb's formula given above, is based on free vortex flow and holds good only for his standard design in which

$$m = 2 \text{ and } \frac{h_2}{D} = \frac{1}{7}$$

where D is the difference of level measured from the minimum water level in the parent channel to the floor of eddy chamber.

MISCELLANEOUS CANAL STRUCTURES

13.15. Cattle Crossings

Whenever canals are to be crossed from one bank to another, certain arrangements must be made for suitable crossings. Normally, bridges are provided across the canals at suitable key points, so as to permit the movement of traffic, cattle, and human beings, etc. across the canals. However, in remote village areas where no major movement is involved, certain arrangements are made so that the cattle, bullock carts, etc. can swim across the canals. Such crossings are called *cattle crossings;* or *cart crossings.*

For canals, carrying smaller water depths, of the order of 0.75 m or so, a ramp is provided for the entry, and another ramp opposite to the entry ramp is provided for the exit. The cattle or cart moves from one bank on the pitched ramp, comes on the bottom of canal within the water, and goes across by walking over the exit ramp.

But for larger canals, having water depth greater than 1.2 m or so, the cattle will have to swim as they cannot straightaway walk down from one bank to another (*via*

water of course). For such canals, therefore, the cattle will swim over the deeper portion of the channel, and be carried downstream by some distance depending upon the width of the channel. A crossing with two ramps on one side and one ramp on the other side located exactly mid way, as shown in Fig. 13.22, will then be necessary. The cattle entering the upper ramp on the side having two ramps will get out at the ramp on the other side : while that entering from there will find exit at the lower ramp on the opposite side, as shown.

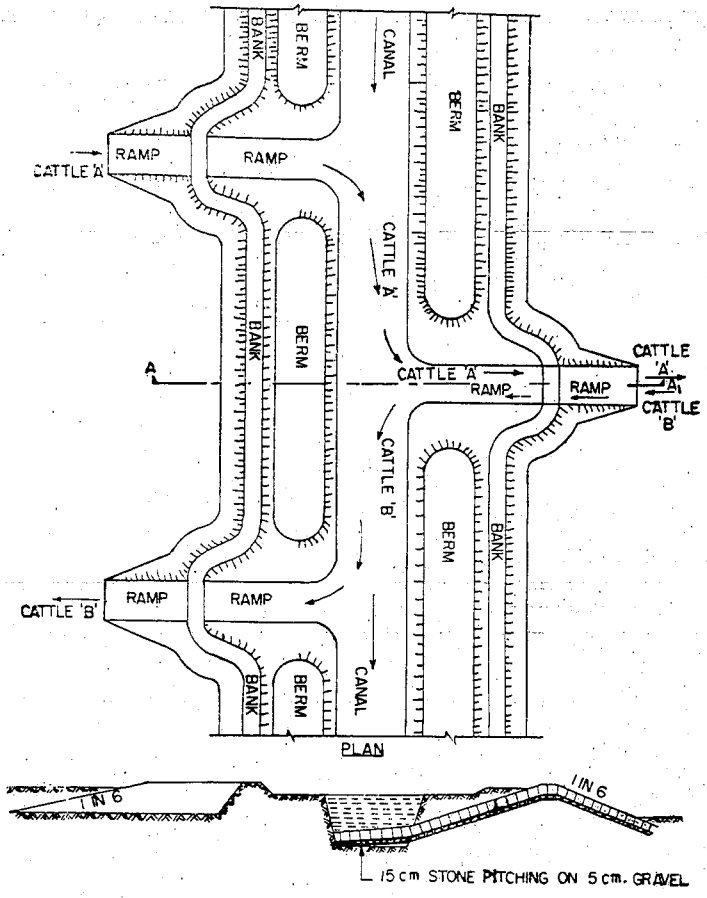


Fig. 13.22. Cattle crossing.

The exact distance to be kept between the two ramps on the same side should theoretically be

$$\frac{2BV}{U} \qquad \text{...(13.31)}$$

where B = width of the channel

V = Av. flow velocity of water in the channel

U = the speed with which cattle swim.

It is better to keep the distance a little longer than shorter, as even if the cattle reach the opposite side much too nearer, they will get at the ramp, but going upstream will be rather difficult.

The cattle crossings though cheaper are generally not preferred, compared to bridges due to the following reasons :

(*i*) they cannot be of much use on canals carrying considerable water depths.

(*ii*) they cannot be used by children or by those human beings who do not know swimming and when channel depth is more than that permitting straight walk over.

(*iii*) the cattle, after crossing one bank and reaching the other, do take rest and ramble about on the bank, causing a lot of damage to the bank.

Cattle crossings, therefore, require regular attention and repairs. They are, therefore, not preferred these days. They may however, be adopted in lone areas, where the likely traffic is very little and the cost of the crossing does not exceed 66% of that of the bridge.

13.16. Bed Bars

Bed bars are those masonry or concrete toe wall like structures which are constructed, at suitable intervals, along an unlined canal, so as to serve as permanent reference marks, and thus to indicate the correct alignment and theoretical bed levels of the canal, at these places.

For smaller canals, the bed bar generally consist of masonry wall partly extending into the bed and flush with it, and partly flush with the bank, as shown in Fig. 13.23. The foundations should be deep and substantial, and should be safe against scouring. The R.L. of the horizontal portion of this wall (which indicates the theoretical bed level of the canal at the point) should be taken by accurate double levelling and recorded on this wall as well as on the L section of the canal for further reference. Such bed bars are usually constructed along minor canals at intervals of 200 m or so.

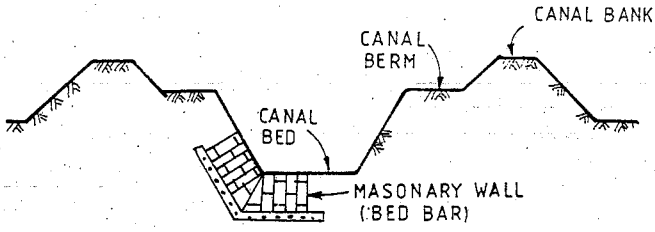


Fig. 13.23. Bed bars for smaller canals.

For larger branch canals, the bed bars usually consist of a masonry or a concrete block provided at suitable intervals along the canal, with their centres at the centre line of the canal, and to flush with the theoretical bed level of the canal at that point, as shown in Fig. 13.24. The depth of the block and its cross-section

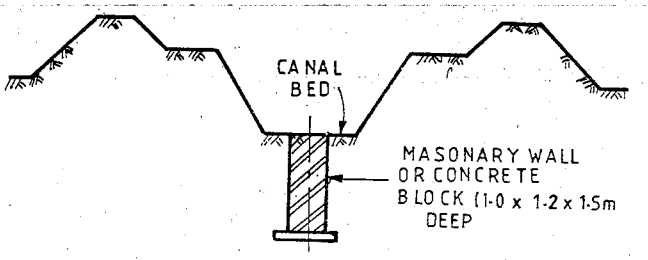


Fig. 13.24. Bed bars for larger branch canals.

should be substantial enough as not to be disturbed by the action of the flowing water. It should also be safe against scouring. A typical size is 1.0 m × 1.2 m × 1.5 m deep block.

The bed bars help in keeping a continuous watch on the behaviour of the canal w.r.t. its silting or scouring tendencies. They also help in regrading or desilting the canal section during its maintenance.

PROBLEMS

1. What is meant by canal regulation ? Enumerate the different engineering structures which may be required to be constructed in a canal project in order to regulate effectively the functioning of the different canals of a canal system ? Discuss briefly the necessity, design and other details of any two such regulation works.

2. (*a*) What is meant by "canal regulation" and what are the different "canal regulation works" ?

(*b*) Describe the necessity and functioning of a "Distributary head regulator" and a "Cross-regulator" in a canal project. Also discuss the procedure that you will adopt for designing these regulation works.

3. Design a cross regulator and a head regulator for a distributary channel taking off from the parent channel, for the following data :

Discharge of the parent channel	= 100 cumecs.
Discharge of the distributary	= 15 cumecs.
FSL of the parent channel	$= \frac{u/s}{d/s} = \frac{208.10}{207.90}$
Bed width of parent channel	$= \frac{u/s}{d/s} = \frac{42\text{ m}}{38\text{ m}}$
Full supply water depth in the parent channel	$= \frac{u/s}{d/s} = \frac{2.5\text{ m}}{2.5\text{ m}}$
FSL of distributary	= 207.10 m
Bed width of distributary	= 15 m
Full depth of water in the distributary	= 1.5 m
Permissible Khosla's safe exit gradient	= 1/6.

4. (*a*) What is meant by "Canal Escapes" ? How do they help in protecting the adjoining areas against flooding due to some breach in the canal embankment ?

(*b*) Describe with neat sketches the different types of canal escapes, that may be constructed on modern canal projects.

5. (*a*) What is a "Metering flume" and how does it help in measuring the discharge of a canal ? Describe with neat sketches, the different types of such flumes and their functioning.

(*b*) What are "modules" and what is their importance in an irrigation canal system ?

6. What are "modules" ? What are the requirements of a good module ? Describe briefly with neat sketches the important types of modules and their suitabilities for a particular project.

7. (*a*) What are 'outlets' ? Enumerate the different types of outlets which are in common use on canal projects. Describe briefly with a neat sketch the functioning, use and design, of an 'Adjustable proportionate module' (APM).

(*b*) What is meant by the term 'Setting of the outlet' ? A semi-modular outlet has a 'Setting' such that its 'flexibility' is equal to 1. Indicate the variations in the discharge of the outlet when the discharge in the parent channel increases or decreases by a certain percentage. (*Engg. Services, 1974*)

[**Ans.** The discharge of the outlet will increase or decrease by the same percentage by which the discharge in the parent channel increases or decreases.]

8. (*a*) What is meant by the terms 'flexibility', 'proportionality', 'setting' and 'sensitivity' as applied to modules. Derive equations for them and discuss relation between these terms.

(*b*) Write note on any two of the following :

(*i*) Flexible module or Semi-modular type canal outlet

(*ii*) Standing wave flume

(*iii*) Canal escapes

(*iv*) Gibb's rigid module

(*v*) Proportional, sub-proportional, and hyper-proportional outlets

(*vi*) APM type outlet

(*vii*) Open flume outlet

(*viii*) Pipe outlets

(*ix*) Kennedy's gauge outlet

(*x*) Cattle crossings

(*xi*) Bed bars .

14

Cross Drainage Works

14.1. Introduction

A cross drainage work is a structure which is constructed at the crossing of a canal and a natural drain, so as dispose of drainage water without interrupting the continuous canal supplies. In whatever way the canal is aligned, such cross drainage works generally become unavoidable. In order to reduce the cross drainage works, the artificial canals are generally aligned along the ridge line called water-shed. When once the canal reaches the watershed line, cross drainage works are generally not required, unless the canal alignment is deviated from the watershed line. However, before the watershed is reached, the canal which takes off from the river has to cross a number of drains, which move from the watershed towards the river, as shown in Fig. 14.1. At all such crossings c_1, c_2, c_3, c_4, etc. cross drainage works are required.

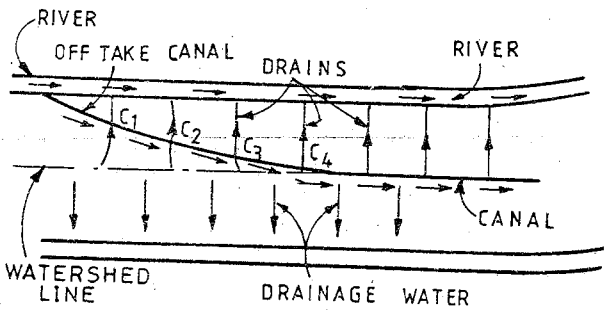


Fig. 14.1

A cross drainage work is generally a costly construction and must be avoided as far as possible. Since a watershed canal* crosses minimum number of drains, such an alignment is preferred to a contour canal which crosses maximum number of drains. The number of cross drainage works may also be reduced by diverting one drain into another and by changing the alignment of the canal, so that it crosses below the junction of two drains.

14.2. Types of Cross-drainage works

The drainage water intercepting the canal can be disposed of in either of the following ways :

(1) By passing the canal over the drainage. This may be accomplished either through (*i*) an *aqueduct;* or through a (*ii*) *syphon-aqueduct.*

(2) By passing the canal below the drainage. This may be accomplished either through (*i*) a *super-passage;* or through a (*ii*) *canal syphon* generally called a *syphon.*

(3) By passing the drain through the canal, so that the canal water and drainage water are allowed to intermingle with each other. This may be accomplished through (*i*) a *level crossing*; or through (*ii*) *inlets and outlets.*

* Also called a ridge canal.

All these different types of cross drainage works are described below in details.

14.2.1. Aqueduct and Syphon Aqueduct. In these works, the canal is taken over the natural drain, such that the drainage water runs below the canal (see Fig. 14.2) either freely or under syphoning pressure. *When the HFL of the drain is sufficiently below the*

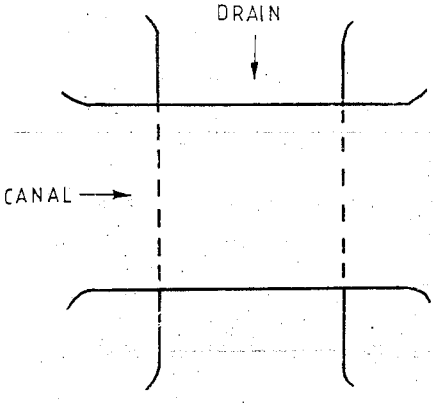


Fig. 14.2. Canal taken over the drain in an aqueduct or a syphon aqueduct (Line Plan of Crossing).

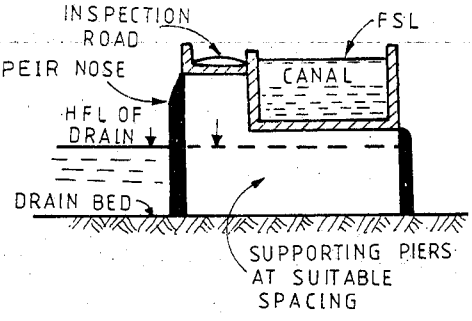


Fig. 14.3. Typical cross-section of an aqueduct.

bottom of the canal, so that the drainage water flows freely under gravity, the structure is known as an Aqueduct (Fig. 14.3). However, if the HFL of the drain is higher than the canal bed and the water passes through the aqueduct barrels under syphonic action, the structure is known as Syphon Aqueduct (See Fig. 14.4).

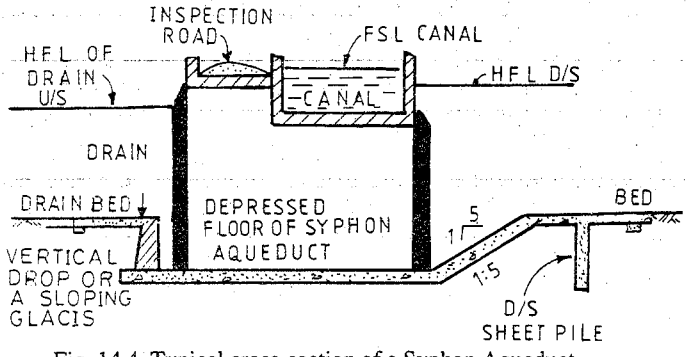


Fig. 14.4. Typical cross-section of a Syphon Aqueduct.

In this type of works, the canal water is taken across the drainage in a trough supported on piers. An inspection road is generally provided alongwith the trough, as shown. An aqueduct is just like a bridge except that instead of carrying a road or a railway, it carries a canal on its top. An aqueduct is provided when sufficient level difference is available between the canal and the natural drainage, and canal bed level is sufficiently higher than the torrent level. In Sirsa, a city near Roper in Punjab, an excellent aqueduct having 20 spans of about 13 m each has been constructed to carry a canal having bed width of 28 metres and a discharge of about 360 cumecs, with a torrent discharge of about 4300 cumecs. A difference of 3.3 metres was available between the bed level of canal and that of torrent in this case.

In the case of a syphon aqueduct, the drain bed is generally depressed and provided with pucca floor, as shown in Fig. 14.4. On the upstream side, the drainage bed may be joined to the pucca floor either by a vertical drop (when drop is of the order of 1 m) or by a glacis of 3 : 1 (when drop is more). The downstream rising slope should not be steeper than 5 : 1.

In this type of cross-drainage works (*i.e.* when the canal is taken over the drainage), the canal remains open to inspection throughout, and the damage caused by floods are rare. However, during heavy floods, the foundations of the work may be susceptible to scour; or waterway of the drain may get choked with debris, trees, etc.

14.2.2. Super-passage and syphon. In these works, the drain is taken over the canal such that the canal water runs below the drain (Fig. 14.5) either freely or under

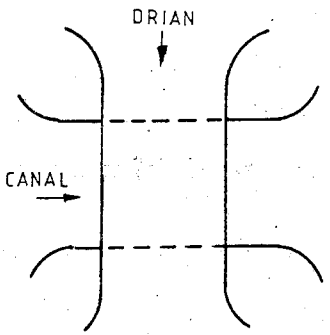


Fig. 14.5. Drain taken over the canal in a Superpassage or in a Syphon. (Line Plan of Crossing)

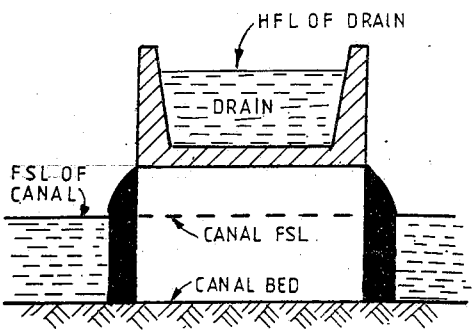


Fig. 14.6. Typical cross-section of a Superpassage.

syphoning pressure. *When the FSL of the canal is sufficiently below the bottom of the drain trough, so that the canal water flows freely under gravity, the structure is known as a Superpassage (Fig. 14.6). However, if the FSL of the canal is sufficiently above the bed level of the drainage trough, so that the canal flows under syphonic action under the trough, the structure is known as a canal syphon or a Syphon (Fig. 14.7).*

A superpassage is thus the reverse of an aqueduct, and similarly, a syphon is a reverse of an aqueduct syphon. However, in this type of cross-drainage works, the inspection road cannot be provided along the canal and a separate bridge is required for the road-way. For affecting economy, the canal may be flumed, but the drainage trough is never flumed.

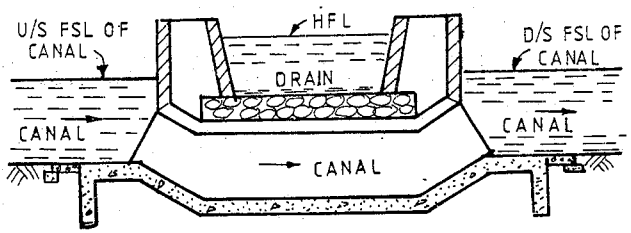


Fig. 14.7. Typical cross-section of a Canal Syphon (generally called a Syphon).

In the case of a syphon, the canal bed is depressed and a ramp is provided at the exit (see Fig. 14.7) so that the trouble of silting is minimised.

14.2.3. Level Crossing. In this type of cross-drainage work, the canal water and drain water are allowed to intermingle with each other. A level crossing is generally provided when a large canal and a huge drainage (such as a stream or a river) approach each other practically at the same level. A typical layout of a level crossing is shown Fig. 14.8.

A regulator is provided across the torrent (drainage) just on the downstream side of the crossing so as to control the discharge passing into the torrent. At the outgoing canal, a regulator is also provided so as control the discharge into the canal. A regulator at the end of the incoming canal is also sometimes required. The arrangement is practically the same as is provided on a canal headworks. This arrangement is generally provided when a huge sized canal crosses a large torrent carrying a very high but short lived* flood discharge. In this arrangement, the perennial drainage discharge is sometimes advantageously used, so as to augment the canal supplies.

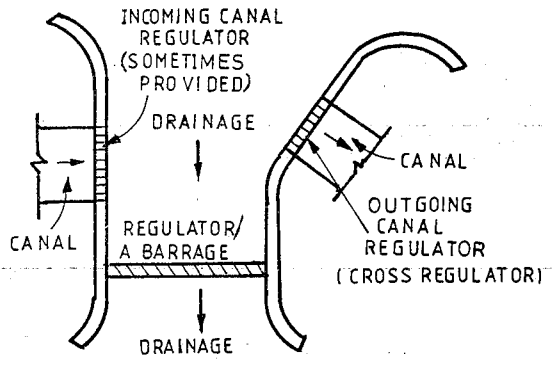


Fig. 14.8. Typical layout of a Level Crossing.

During dry season, when there are no floods, the torrent regulator is generally kept closed and the outgoing canal regulator is kept fully open, so that the canal flows without any interruption. During floods, however, the torrent regulator is opened so as to pass the flood discharge. A beautiful level crossing has been provided in Eastern U.P. under Sarda Sahayak Pariyojna ; where a canal carrying 370 cumecs crosses the Sarda river carrying a high flood discharge of the order of 10,000 cumecs.

14.2.4. Inlets and Outlets. An inlet is a structure constructed in order to allow the drainage water to enter the canal and get mixed with the canal water and thus to help in augmenting canal supplies. Such a structure is generally adopted when the drainage discharge is small and the drain crosses the canal with its bed level equal to or slightly higher than the canal F.S.L. Moreover, for the canal to remain in regime, the drain water must not admit heavy load of silt into the canal. Thus, in an inlet, the drainage water is simply added to the canal.

But, when the drainage discharge is high or if the canal is small, so that the canal section cannot take the entire drainage water, an outlet may sometimes be constructed to escape out the additional discharge at a suitable site, a little downstream along the canal. It is not necessary that the escaped discharge should be equal to the admitted discharge.

Similarly, it is also not necessary, that the number of inlets and outlets should be the same. There may be one outlet for two or three inlets. The outlet is generally combined with some other work where arrangement for escaping is in any case to be provided or may be added at a small extra cost.

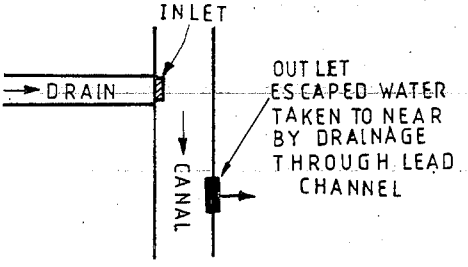


Fig. 14.9. Inlet and outlet (Plan).

An *inlet* essentially consists of an open cut in a canal bank, suitably protected by pitching, to admit the upland drainage

* When floods are intermittent and not continuous.

water into the canal. The bed and sides of the canal are also pitched for a certain distance upstream and downstream of the inlet. Similarly the *outlet* is another open cut in the canal bank with bed and sides of the cut properly pitched. The escaping water from the outlet is taken away by a lead channel to some nearby drain, on the downstream side of the surface outlet.

This type of cross-drainage work (*i.e.* those requiring intermingly of canal water with drainage water) are inferior to aqueduct or superpassage type of works, but they are cheaper. Hence, the aqueduct or superpassage type of works are generally used when high flood drainage discharge is large and continues for a sufficient time. A level crossing is used when the high flood drainage discharge is large but short lived. Inlets and outlets are used when the high flood drainage discharge is small.

14.3. Selection of a Suitable Type of Cross-Drainage Work

The relative bed levels, water levels, and discharge of the canal and the drainage are the primary factors which govern and dictate the type of cross drainage work that may prove to be most suitable at a particular place. For example, if the bed level of the canal is sufficiently above the HFL of the drain, an aqueduct is the first and obvious choice. But, if the bed level of the drain is sufficiently above the canal FSL, a superpassage may be constructed. Similarly, when a canal carries a small discharge compared to the drain, the canal may be taken below the drain by constructing a syphon, as against a syphon aqueduct which is adopted when the drain with smaller discharge can be taken below a large canal.

However, in actual field, such ideal conditions may not be available and the choice would then depend upon many other factors, such as :

(*i*) Suitable canal alignment.

(*ii*) Nature of available foundation.

(*iii*) Position of watertable and availability of dewatering equipment.

(*iv*) Suitability of soil for embankment.

(*v*) Permissible head loss in canal.

(*vi*) Availability of funds.

The relative bed levels of the canal and the drainage may be changed and manipulated by suitably changing the canal alignment, so that the point of crossing is shifted upstream or downstream of the drainage. For example, if the canal alignment is such that sufficient headway is not available between HFL of drain and bed of the canal, (although canal bed is higher) a syphon aqueduct has to be normally adopted. But, however, if other conditions (enumerated above) are not favourable for the construction of a syphon-aqueduct, the canal alignment may be changed so that the crossing is shifted to the downstream where drainage bed is low and thus sufficient headway becomes available for constructing an aqueduct in place of syphon aqueduct. The canal alignment is, therefore, finalised only after finalising the cross drainage works.

Compared to an aqueduct, a superpassage is inferior and should be avoided whenever possible. Similarly, a syphon-aqueduct (unless large drop in drainage bed is required) is superior to a syphon. A level crossing may become inevitable in certain cases. For example, when a large canal crosses a large torrent at almost equal bed levels, a level crossing may remain to be the only answer. An inlet may be adopted when a small drain crosses the canal with its bed level equal to canal FSL or slightly higher than it

(*i.e.* bed conditions similar to those favouring the choice of a syphon or a superpassage). Inlets, though cheaper, are not preferred these days because their performance has not been very satisfactory.

14.4. Various Types of Aqueducts and Syphon-Aqueducts

They may be classified into three types depending on the sides of the aqueduct :

Type I. In this type, the sides of the aqueduct are earthen bank with complete earthen slopes. The length of the culvert through which the drainage water has to pass under the canal should not only be sufficient to accommodate the water section of the canal but also the earthen banks of the canal with adequate slopes (Fig. 14.10).

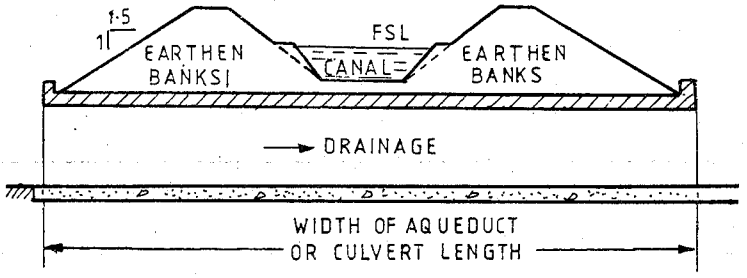


Fig. 14.10. Aqueduct (Type I)

Type II. In this type, the canal continues in its earthen section over the drainage, but the outer slopes of canal banks are replaced by retaining walls, thereby, reducing the length of the drainage culvert by that much extent (Fig. 14.11).

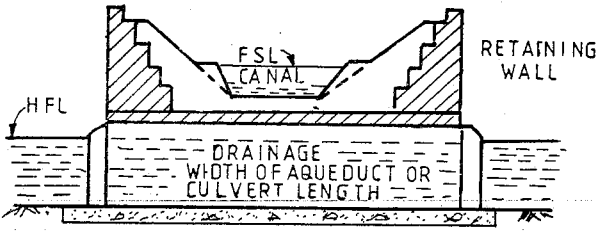


Fig. 14.11. Aqueduct (Type II)

Type III. In this type, earthen section of the canal is discontinued and the canal water is carried in a masonry or a concrete trough. The canal is generally flumed in this case, so as to effect economy in construction.

The culvert length or width of aqueduct is maximum in Type I and minimum in Type III. An intermediate value exists in Type II.

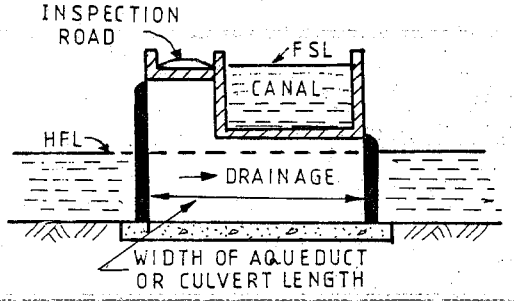


Fig. 14.12. Aqueduct (Type III)

Selection of the Suitable Type. *The selection of a particular type out of three types of aqueducts or syphon-aqueducts lies on the considerations of economy. The cheapest of the three types at a particular place shall be the obvious choice.*

In fact, in all cases, the cost of abutments and wing walls, is independent of the length of the culvert along the canal. In type I, no canal wings are required since the canal section is not at all changed. However, in this type, the width of the aqueduct is

the largest. Type I will, therefore, prove economical only where the length of the aqueduct is small and where the cost of bank connections would be large in comparison to the savings obtained from the reduction in the width of the aqueduct.

In type III, the width of the aqueduct is minimum but the cost of bank connections is maximum. This type is, therefore, suitable where the length of the aqueduct is very large and where the cost of bank connections would be small in comparison to the savings obtained from the reduction in the width of the aqueduct.

On the basis of above discussion, it can be concluded that the choice of a particular type depends mainly upon the length of the aqueduct (i.e. the width of the drainage) in relation to the size of the canal. On a very small drain, type III is most economical; while on a very wide drainage, type I is most economical. Type II is intermediate between type I and type III. The exact choice of a particular type in a particular case can be made by working out the cost of all the types and then choosing the cheapest.

14.5. Design Considerations for Cross Drainage Works

The following steps may be involved in the design of an aqueduct or a syphon-aqueduct. The design of a superpassage and a syphon is done on the same lines as for aqueducts and syphon aqueducts, respectively, since hydraulically there is not much difference between them, except that the canal and the drainage are interchanged by each other.

14.5.1. Determination of Maximum Flood Discharge. The high flood discharge for smaller drains may be worked out by using empirical formulas ; and for large drains, other reliable methods such as Hydrograph analysis, Rational formula, etc. may be used.

14.5.2. Fixing the Waterway Requirements for Aqueducts and Syphon-Aqueducts. An approximate value of required waterway for the drain may be obtained by using the Lacey's equation, given by

$$P = 4.75 \cdot \sqrt{Q}$$

where P = is the watted perimeter in metres

Q = Total discharge in cumecs.

For wide drains, the wetted perimeter may be approximately taken equal to the width of the drain and hence, equal to the waterway required. However, no extra provision is generally made for the space occupied by piers. Hence, if the total waterway provided is equal to P, the effective or clear waterway will be less than P by as much extent as is occupied by pier widths. For smaller drains, a smaller figure for the waterway than that given by Lacey's regime perimeter, may be chosen. The maximum permissible reduction in waterway from Lacey's perimeter is 20%. Hence, for smaller drains, the width of the waterway provided should be so adjusted as to provide this required perimeter (minimum value $= 0.8\,P$). The decided clear water way width is provided in suitable number of bays (spans).

Size of the Barrels. After having fixed the waterway width & number of compartments (bays), the height of the drain barrels has to be fixed. In case of an aqueduct, the canal trough is carried clear above the drain HFL, and drain bed is not to be depressed. Hence, the height of bay openings is automatically fixed in aqueducts, as equal to the difference between HFL and DBL of drain.

However, in syphon-aqueducts, the required area of the drainage waterway can be obtained by dividing the drainage discharge by the permissible velocity through the barrels. This velocity through the barrels is generally limited to 2 to 3 m/sec. The waterway area is then divided by the decided waterway width of the drain openings, to compute the height of the openings, and the extent of depressed floor.

Due to the reduction in the width of the drainage, afflux is produced near the work site. The afflux will increase more and more, if the waterway is reduced more and more. The value of afflux is limited, so that there is no flooding of the country-side. The afflux may be calculated by using Unwin's formula as explained below in the following article.

14.5.3. Afflux and Head Loss through Syphon Barrels. It was stated earlier that the velocity through syphon barrels is limited to a scouring value of about 2 to 3 m/sec. A higher velocity may cause quick abrasion of the barrel surfaces by rolling grit, etc. and shall definitely result in higher amount of afflux on the upstream side of the syphon or syphon-aqueduct, and thus, requiring higher and longer marginal banks.

The head loss (h) through syphon barrels and the velocity (V) through them are generally related by Unwin's formula*, given as :

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2}{2g} - \frac{V_a^2}{2g} \qquad ...(14.1)$$

where L = Length of the barrel.

R = Hydraulic mean radius of the barrel.

V = Velocity of flow through the barrel.

V_a = Velocity of approach and is often neglected.

f_1 = Coefficient of head loss at entry

= 0.505 for unshaped mouth

= 0.08 for bell mouth.

f_2 = is a coefficient such that the loss of head through the barrel due to surface friction is given by

$f_2 \cdot \frac{L}{R} \cdot \frac{V^2}{2g}$; where f_2 is given as :

$$f_2 = a\left(1 + \frac{b}{R}\right) \qquad ...(14.2)$$

where the values of a and b for different materials may be taken as given in Table 14.1.

Table 14.1

Material of the surface of barrel	a	b
Smooth iron pipe	0.00497	0.025
Encrusted pipe	0.00996	0.025
Smooth cement plaster	0.00316	0.030
Ashlar or brick work	0.00401	0.070
Rubble masonry or stone pitching	0.00507	0.250

*The total head loss consists of three losses, *i.e.*

Entry loss $= f_1 \frac{V^2}{2g}$, (*ii*) Friction loss $= \frac{f_2 L V^2}{2gR}$, (*iii*) Exit loss $= \frac{V^2}{2g}$.

After having fixed the velocity (V) through the barrels, the head (h) required to generate that much velocity can be found by using the equation (14.1).

The d/s HFL of the drain remains unchanged by the construction of works, and thus the u/s HFL can be obtained by adding h to the d/s HFL. The u/s HFL, therefore, gets headed up by an amount equal to h and is known as afflux. The amount of afflux is limited because the top of guide banks and marginal bunds, etc. are governed by this raised HFL. So a limit placed on afflux will limit the velocity through the barrels and *vice versa*. Hence, by premitting a higher afflux and, therefore, a higher velocity through the barrels, the cross-sectional area of syphon barrels can be reduced, but there is a corresponding increase in the cost of guide banks and marginal bunds and also the length of d/s protection is increased. Hence, an economic balance should be worked out and a compromise obtained between the barrel area and afflux. Moreover, in order to reduce the afflux for the same velocity, the entry is made smooth by providing bell mouthed piers and surface friction is reduced by keeping the inside surface of the barrels as smooth as possible.

14.5.4. Fluming of the Canal. The contraction in the waterway of the canal (*i.e.* fluming of the canal) will reduce the length of barrels or the width of the aqueduct. This is likely to produce economy in many cases. The fluming of the canal is generally not done when the canal section is in earthen banks. Hence, the canal is generally not flumed in works of Type I and Type II. However, fluming is generally done in all the works of Type III.

The maximum fluming is generally governed by the extent that the velocity in the trough should remain subcritical (of the order of 3 m/sec). Because, if supercritical velocities are generated, then the transition back to the normal section on the downstream side of the work may involve the possibility of the formation of a hydraulic jump. This hydraulic jump, where not specifically required and designed for, would lead to undue loss of head and large stresses on the work. The extent of fluming is further governed by the economy and permissible loss of head. The greater is the fluming, the greater is the length of transition wings upstream as well as downstream. This extra cost of transition wings is balanced by the saving obtained due to the reduction in the width of the aqueduct. Hence, an economic balance has to be worked out for any proposed design.

After deciding the normal canal section and the flumed canal section, the transition has to be designed so as to provide a smooth change from one stage to the other, so as to avoid sudden transition and the formation of eddies, etc. For this reason, the u/s or approach wings should not be steeper than $26\frac{1}{2}^\circ$ (*i.e.* 2 : 1 splay) and the d/s or departure wings should not be steeper than $18\frac{1}{2}^\circ$ (*i.e.* 3 : 1 splay). Generally, the normal earthen canal section is trapezoidal, while the flumed pucca canal section is rectangular. It is also not necessary to keep the same depth in the normal and flumed sections. Rather, it may sometimes be economical to increase the depth and still further reduce the channel width in cases where a channel encounters a reach of rocky terrain and has to be flumed to curtail rock excavation. But an increase in the water depth in the canal trough will certainly increase the uplift pressures on the roof as well as on the floor of the culvert, thus requiring larger roof and floor sections and lower foundations. Due to these reasons, no appreciable economy may be obtained by increasing the depth.

The following methods may be used for designing the channel transitions :

(*i*) Mitra's method of design of transitions (when water depth remains constant).

(*ii*) Chaturvedi's method of design of transitions (when water depth remains constant).

(*iii*) Hind's method of design of transitions (when water depth may or may not vary).

(*i*) **Mitra's Hyperbolic Transition when water depth remains constant.** Shri A.C. Mitra, Chief Engineer, U.P. Irrigation Deptt. (Retd.), has proposed a hyperbolic transition for the design of channel transitions. According to him, the channel width at any section *X–X*, at a distance *x* from the flumed section (Fig. 14.13) is given by

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - (B_n - B_f)\, x} \qquad ...(14.3)$$

where B_n = Bed width of the normal channel section.

B_f = Bed width of the flumed channel section.

B_x = Bed width at any distance x from the flumed section.

L_f = Length of transition.

Derivation of equation (14.3) is given below :

The above transition equation (*i.e.* equation 14.3) was derived on the basis that the rate of change of velocity per unit length of the transition remains constant throughout

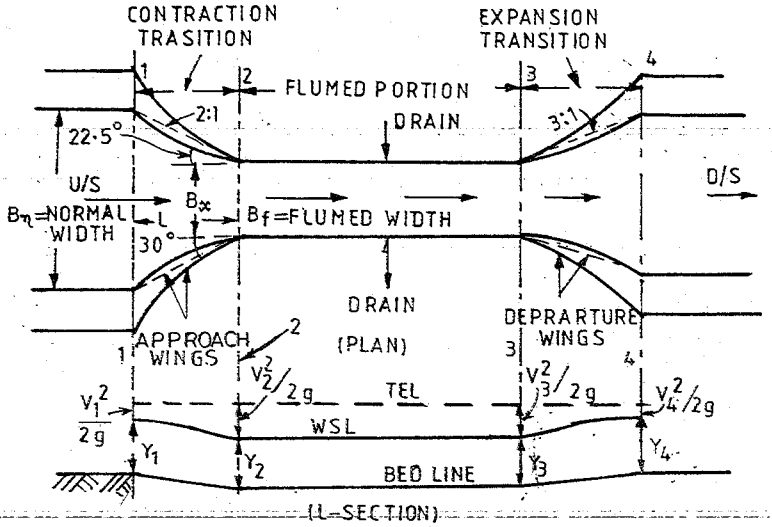


Fig. 14.13

the transition length. Thus, if V_n, V_f and V_x represent velocities at the corresponding sections, we have

$$\frac{V_f - V_x}{x} = \frac{V_f - V_n}{L_f} \qquad ...(i)$$

Now, since depth y is assumed to be constant and the total discharge Q is also constant, we have

Velocity × Area = Discharge

$\therefore V_f \cdot B_f \cdot y = V_x \cdot B_x \cdot y = V_n \cdot B_n \cdot y = Q$ (assuming rectangular section throughout).

$\therefore \quad V_f \cdot B_f = V_x \cdot B_x = V_n \cdot B_n$

$$= \frac{Q}{y} = \text{constant} = K \text{ (say)}$$

or $$V_f = \frac{K}{B_f}$$

$$V_x = \frac{K}{B_x}$$

$$V_n = \frac{K}{B_n}$$

Substituting these values in equation (i) we get

$$\left[\frac{\frac{K}{B_f} - \frac{K}{B_x}}{x}\right] = \left[\frac{\frac{K}{B_f} - \frac{K}{B_n}}{L_f}\right]$$

or $$\frac{B_x - B_f}{B_f \cdot B_x \cdot x} = \frac{B_n - B_f}{L_f \cdot B_f \cdot B_n}$$

or $$B_x \cdot L_f B_f \cdot B_n - L_f \cdot B_f^2 \cdot B_n = B_n B_f B_x \cdot x - B_f^2 B_x \cdot x$$

or $$B_x \cdot L_f \cdot B_f \cdot B_n - B_n \cdot B_f \cdot B_n \cdot x + B_f^2 \cdot B_x \cdot x = L_f \cdot B_f^2 \cdot B_n$$

or $$B_x \cdot B_f \left[L_f \cdot B_n - B_n \cdot x + B_f \cdot x\right] = L_f \cdot B_f^2 \cdot B_n$$

or $$B_x \left[L_f \cdot B_n - x\,(B_n - B_f)\right] = L_f \cdot B_f \cdot B_n$$

or $$B_x = \left[\frac{L_f \cdot B_f \cdot B_n}{L_f \cdot B_n - x\,(B_n - B_f)}\right] \qquad \ldots(14.3)$$

This is the required equation (14.3).

(ii) **Chaturvedi's Semi-Cubical Parabolic Transition when water depth remains constant.** Prof. R.S. Chaturvedi, Head of Civil Engineering Deptt. in Roorkee University (Retd.), on the basis of his own experiments, had in 1963, proposed the following equation for the design of channel transitions when water depth remains constant.

$$x = \frac{L \cdot B_n^{3/2}}{B_n^{3/2} - B_f^{3/2}} \left[1 - \left(\frac{B_f}{B_x}\right)^{3/2}\right] \qquad \ldots(14.4)$$

Choosing various convenient values of B_x, the corresponding distance x can be computed easily from the above equation.

(iii) **Hind's Method for the design of Transitions when water depth may also vary.** This is a general method and is applicable either when the depth in the flumed and unflumed portions are the same, or when these depths are different.

In Fig. 14.13, the contraction transition (*i.e.* the approach transition) starts at section 1-1 and finishes at section 2-2. The flumed section continues from section 2-2 to section 3-3. The expansion transition starts at section 3-3 and finishes at section 4-4. From

section 4-4 onwards, the channel flows in its normal cross-section and the conditions at this section are completely known. Let V and y with appropriate subscripts refer to velocities and depths at different sections.

The FSL at section 4–4 = Bed level at section 4–4 + y_4 = (known)

$\therefore$ TEL at section 4–4 = FSL at section 4–4 + $\frac{V_4^2}{2g}$ = (known)

Between section 3-3 and 4-4, there is an energy loss in the expansion, which is generally taken as equal to $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$.

$\therefore$ TEL at section 3–3 = TEL at section 4–4 (known) + $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

As the trough dimensions at section 3-3 are known, V_3 is also known, and hence, TEL at section 3-3 can be computed. Knowing TEL at 3-3 ; FSL at 3-3 can be calculated by subtracting $\frac{V_3^2}{2g}$ from TEL. Similarly, bed level at 3-3 can also be computed by substracting y_3 from FSL at 3-3.

Between sections 2-2 and 3-3, the channel flows in a trough of constant cross-section. The only loss in the trough (H_L) is the friction loss which can be computed with Manning's formula, *i.e.*,

$$\left(Q = \frac{1}{n} A \cdot R^{2/3} \cdot S^{1/2}\right)$$

or $$Q = \frac{1}{n} A \cdot R^{2/3} \cdot \sqrt{\frac{H_L}{L}}$$

or $$H_L = \frac{Q^2 \cdot n^2 \cdot L}{A^2 \cdot R^{4/3}}.$$

Adding this head loss H_L to TEL of section 3-3, the TEL at section 2-2 is obtained. The FSL at section 2-2 can then be obtained by subtracting $\frac{V_2^2}{2g}$ from TEL of 2-2. Similarly, the bed level at section 2-2 can be easily obtained by further substracting y_2 from FSL at 2-2. Since the depth and velocity are constant in the trough, the TEL, FSL and bed lines are all parallel to each other from section 2-2 to 3-3.

Between section 1-1 and 2-2, there is a loss of energy due to contraction. This loss is generally taken as equal to $0.2\left[\frac{V_2^2 - V_1^2}{2g}\right]$.

Thus the TEL at section 1-1

$$= \text{TEL at section } 2\text{–}2 + 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right) = \text{(known)}.$$

Knowing TEL at section 1-1, FSL at 1-1 can be obtained by subtracting $\frac{V_1^2}{2g}$ from TEL at 1-1. Similarly, bed level at 1-1 can be obtained by subtracting y_1 from FSL at 1-1.

The bed level, FSL and TEL having been determined at all the four sections, the total energy line may be drawn by assuming it to be a straight line between adjacent sections. The bed line may also be drawn straight between adjacent sections, provided, the rise or fall in bed is small. The corners should, however, be rounded off in this case. However, if the change in bed level is considerable, the bed line in the transition section should be drawn as a smooth reverse curve, tangential to the bed lines at ends.

Water surface in Transition : In the contraction transition between section 1-1 to 2-2, there will be a drop in water surface due to the drop in energy line and also due to the increased velocity head at 2-2. This drop in water surface has to be negotiated by a smooth curve tangential at both ends. This can be easily accomplished by using two parabolic curves meeting tangentially at the centre of the transition, as shown in Fig. 14.14.

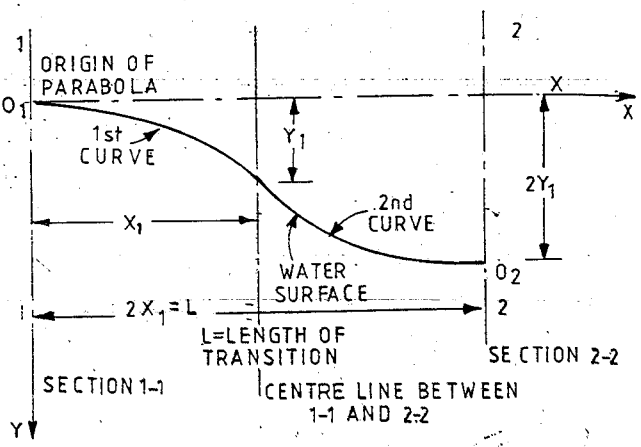


Fig. 14.14. Water Surface profile for Transition Contraction.

Let $2X_1 = L$ = The length in which fluming has been done.

$2Y_1$ = Total difference in water levels between section 1-1 and 2-2.

The distance of the middle point of transition will be X_1 and drop in water surface will be Y_1. The equation of the first parabolic curve, with origin at water surface of section 1-1 (*i.e.* O_1) is given by

$$Y = C \cdot X^2$$

when $Y = Y_1$, and $X = X_1$

$$C = \frac{Y_1}{X_1^2}$$

Therefore, the equation of parabola becomes

$$Y = \left[\frac{Y_1}{X_1^2}\right] X^2 \qquad ...(14.5)$$

Using the above equation, the first parabolic curve can be easily plotted. Similarly, the second parabolic curve can be plotted by taking the origin at O_2 on section 2-2.

The water surface in the expansion transition between sections 3-3 and 4-4 can also be plotted in a similar fashion, where there will be a rise in the water surface from section 3-3 to 4-4, as shown in Fig. 14.13.

After having plotted the water surface profile over the entire length, the velocity head say (h_v) can be found by measuring the vertical distance between TEL and water surface line at any point. The velocity head can then be converted into equivalent velocity by using $V = \sqrt{2g \cdot h_v}$. Hence, the velocity at each point can be known. The

cross-sectional area required to pass the given discharge at each point can be found by dividing discharge by velocity at that point $\left(i.e.\ A = \frac{Q}{V} \right)$

In trapezoidal channel of water depth y, the bed width B, and side slopes $s : 1$; area is given by

$$A = BD + s \cdot y^2. \qquad ...(14.6)$$

In flared wings, the side slopes are generally brought to vertical from an initial slope of $s : 1$ and, therefore, the side slopes at any point can be interpolated in proportion to the length of transition undergone. Thus at any point A, y and s are known and hence the value of B can be worked out at this pt. by using the equation (14.6). The width of the canal at various points in the transition can thus be determined. Hence, all the dimensions of the transition are fully found out.

14.5.5. Design of Pucca Canal Trough. The canal trough is designed as follows :

For an Aqueduct. In case of an aqueduct, the bottom of the canal *i.e.* the roof of the culvert is subjected to the dead weight and the vertical load of water from the top, as shown in Fig. 14.15.

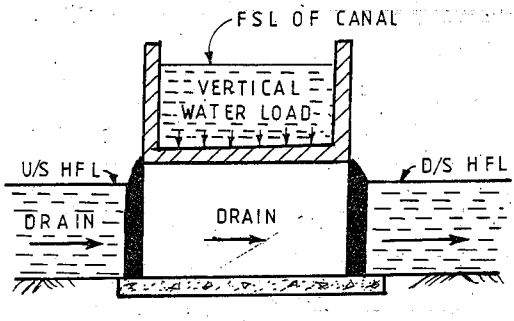


Fig. 14.15

Since in an aqueduct, there is no uplift from the underside acting on the bottom of canal bed, the canal bottom has to be designed for taking the dead weight and full water load (when canal is running full) by either providing a thickness sufficient to take this much of load merely by gravity or by providing a reinforced concrete slab with reinforcement at its bottom.

The side walls of the canal are to be designed as retaining walls. They may be made of masonry or R.C.C. It is preferable to have an entire R.C.C. section. The retaining walls will be designed to carry the entire horizontal force exerted by the canal water and shall, therefore, carry reinforcement on the water face.

For an Aqueduct Syphon. In case of an aqueduct syphon, besides the vertical load of canal water, one more force comes into action *i.e.* the uplift pressure exerted by the drain water. The roof of the culvert *i.e.* the bottom-slab of canal should now be designed to withstand these two forces independently. Although these two forces act in a opposition to each other but still, under the worst circumstances, there may be times when only one of them may be acting. For example, when drainage is flowing at its maximum high flood level, canal may be empty. Similarly, there may not be any drainage water touching the slab when the canal may be running full. Hence, the slab should be designed for :

(*i*) full water load and dead weight, with no uplift

(*ii*) full-uplift with no water load.

The amount of the uplift pressures exerted by the drain water on the roof of the culvert can be evaluated by drawing the hydraulic gradient line (H.G. line), as shown Fig. 14.16.

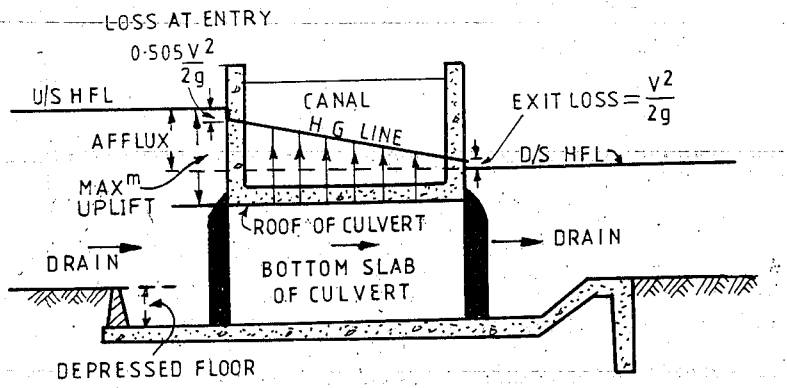


Fig. 14.16

The uplift pressure at any point under the roof of the culvert will be equal to the vertical ordinate between the hydraulic gradient line and the underside of the canal trough at that point, as shown in Fig. 14.16. From this uplift diagram of Fig. 14.16, it is very evident that the maximum uplift occurs at the upstream end point near the entry. The uplift pressure on the underside of the trough at the upstream end will be =[u/s water level – Entry loss – the level of the underside of the trough]. The slab thickness should be designed to withstand this maximum uplift.

When the slab is designed to counter-balance the maximum uplift, merely be gravity, it is sometimes found that the slab thickness required is less than what is required for the first condition (*i.e.* when designed for full water load). But many a times, the thickness required for balancing uplift may exceed the thickness required for balancing the water load. In that case, it is generally not advisable to increase the thickness because any increase in thickness will result in lowering the levels of both the roof of the culvert as well as the bottom slab of culvert. This, in turn, will increase the uplift on the roof slab as well as on the bottom slab of culvert. Hence, in such a case, the thickness of the roof slab is generally provided from the considerations of water load and the remaining uplift is resisted by bending by providing top reinforcement in the roof slab. In such a case, the roof will have to be anchored to the bottom slab through piers by steel bolts, etc. so as to provide necessary end reactions for upward bending. A typical anchoring arrangement is shown in Fig. 14.17. *A better alternative may be to provide full fledged R.C.C. box culvert.*

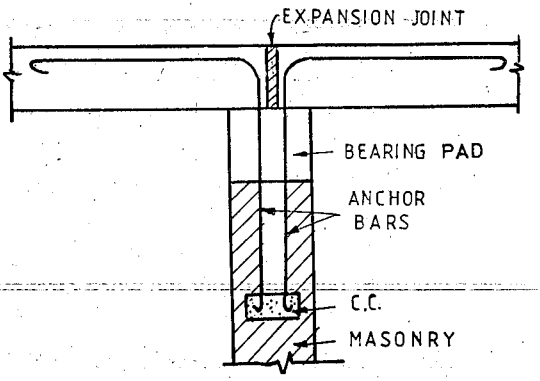


Fig. 14.17. Typical details of anchoring.

14.5.6. Design of Bottom Floor of Aqueduct and Syphon Aqueduct. The floor of the aqueduct or syphon-aqueduct is subjected to uplift due to two causes :

(*a*) *Uplift due to water-table.* This force acts where the bottom floor is depressed below the drainage bed, especially in syphon aqueducts.

The maximum uplift under the worst condition would occur when there is no water flowing in the drain and the watertable has risen up to the drainage bed. The maximum net uplift in such a case would be equal to the difference in level between the drainage bed and the bottom of the floor, as shown in Fig. 14.18.

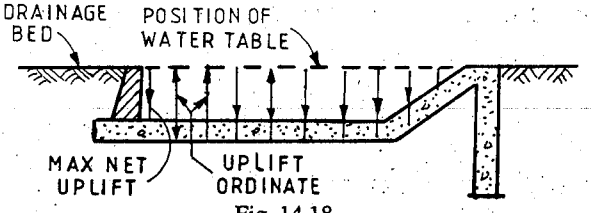


Fig. 14.18

(*b*) *Uplift due to seepage of water from the canal to the drainage.* The maximum uplift due to this seepage occurs when the canal is running full and there is no water in the drain. The computations of this uplift, exerted by the water seeping from the canal on the bottom floor, is very complex and difficult, due to the fact that the flow takes place in three dimensional flownet. The flow cannot be approximated to a two dimensional flow, as there is no typical place across which the flow is practically two dimensional. Hence, for the smaller works, Bligh's Creep theory may be used for assessing the seepage pressures. But, for the larger works, the uplift pressures must be checked by model studies. The seepage pressure can be evaluated by Bligh's theory as explained below :

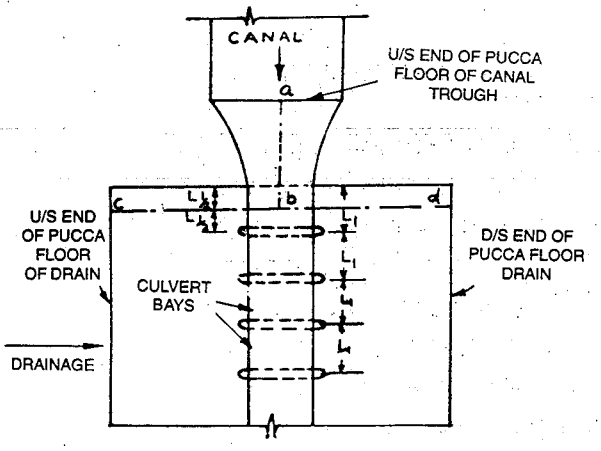


Fig. 14.19

The seepage flow occurs from the beginning of pucca canal trough (point *a*) and reappears in the drainage bed on either side of the impervious floor along the centre of the floor of the first culvert bay (say point *c* or point *d*) in Fig. 14.19. Point *b* is the point under the centre of the floor of the first culvert bay.

The seepage path from *a* to *b* and from *b* to *c* can be known. The total creep length will then be equal to $= ab + bc$. If H is the total seepage head (*i.e.* H = FSL of canal – d/s bed level of drain), the residual head at the point *b* (*i.e.*, H_b) is then given by Bligh's theory as equal to

$$H_b = H - \left[\frac{H}{ab+bc} \times ab\right]$$

or

$$H_b = \left[\frac{H}{ab+bc} \times bc\right] \qquad ...(14.7)$$

The floor of the syphon-aqueduct must be designed for the total uplift which is equal to the sum of the uplift due to seepage plus the uplift due to static head. The total uplift may be partly resisted by the wt. of the floor and partly by bending in reinforcement.

Methods of reducing uplift on the floor. The uplift on the bottom floor may be reduced in two ways :—

(*i*) By extending the impervious canal trough on either side of the drainage so as to increase the creep length *ab*. A puddle apron may be used in place of concrete floor, if clay is easily available.

(*ii*) By providing drainage holes in the culvert floor so as to release the uplift. If such relief holes are provided in the bottom floor ; an inverted filter, should also be provided below the floor. The inverted filter would help in preventing the soil particles from getting out of the holes. The performance of such holes may not prove very successful in actual field as it appears to be on paper. Because, if these get choked or if there occurs some defect in filter system, there may be a danger of failure of work by excessive uplift or by undermining.

14.5.7. Design of Bank Connections. Two set of wings are required in aqueducts and syphon-aqueducts. These are :

(*i*) Canal Wings or Land Wings.

(*ii*) Drainage Wings or Water Wings.

(*i*) **Canal wings or Land wings.** These wings provide a strong connection between the masonry or concrete sides of a canal trough and earthen canal banks. These wings are generally warped in plan so as to change the canal section from trapezoidal to rectangular. They should be extended upto the end of splay. These wings may be designed as retaining walls for maximum differential earth pressure likely to come on them with no water in the canal. The foundations of these wings should not be left on filled earth. They should be taken deep enough to give safe creep length.

(*ii*) **Drainage wings or Water wings or River wings.** These wing walls retain and protect the earthen slopes of the canal, guide the drainage water entering and leaving the work, and join it to guide banks and also provide a vertical cut-off for the water seeping from the canal into the drainage bed. The foundations of these wing walls should be taken below the deepest anticipated scour in the river. The sections of these wing walls should be capable of withstanding the maximum differential earth pressure likely to come on them.

The layouts of these sets of wings depend on the extent of contraction of canal and drainage waterways, and the general arrangement of the work.

Design Examples

Example 14.1. *Design a suitable cross-drainage work, given the following data at the crossing of a canal and a drainage.*

Canal

Full supply discharge	$= 32$ *cumecs*
Full supply level	$=$ *R.L. 213.5*
Canal bed level	$=$ *R.L. 212.0 m.*
Canal bed width	$= 20.$
Trapezoidal canal section with $1\frac{1}{2} H : 1 V$ *slopes.*	
Canal water depth	$= 1.5$ *m.*

Drainage

High flood discharge = *300 cumecs.*
High flood level = *210.0 m.*
High flood depth = *2.5 m.*
General ground level = *212.5 m.*

Solution. Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (212.0 m) is much above the H.F.L. of drainage (*i.e.* 210.0 m) an **aqueduct** will be constructed. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For effecting economy, the canal shall be flumed.

Step 1. Design of Drainage Waterway

Lacey's regime perimeter $= P = 4.75\sqrt{Q}$

where Q = High flood discharge of drain
= 300 cumecs (given)

$P = 4.75 \cdot \sqrt{300} = 82.3$ m.

Let the clear span between piers be 9 m and the pier thickness be 1.5 m.

Using 8 bays of 9 m each, clear waterway $= 8 \times 9 = 72$ m.

Using 7 piers of 1.5 each, length occupied by piers $= 7 \times 1.5 = 10.5$ m.

Total length of waterway $= 72 + 10.5 =$ **82.5 m**

Step 2. Design of Canal Waterway

Bed width of canal = 20.0 m.

Let the width be flumed to 10.0 m.

Providing a splay of 2 : 1 in contraction, the length of contraction transition

$$= \frac{20 - 10}{2} \times 2 = \mathbf{10.0\ m}$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{20 - 10}{2} \times 3 = \mathbf{15\ m}$$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m (provided).

In transitions, the side slopes of the canal section will be warped in plan from the original slope of $1\frac{1}{2}$: 1 to vertical.

Step 3. Head loss and bed levels at different sections. (Fig. 14.20).

At Section 4-4

At section 4-4, where the canal returns to its normal section, we have

Area of trapezoidal canal section

$= (B + 1.5y)\, y$

$= (20 + 1.5 \times 1.5)\ 1.5 = 22.5 \times 1.5 = 33.75\ \text{m}^2$

$$\text{Velocity} = V_4 = \left(\frac{Q}{A}\right) = \frac{32}{33.75} = \mathbf{0.947\ m/sec}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9.81} = 0.046\ \text{m}$$

R.L. of bed at 4-4 = **212.0 m** (given)

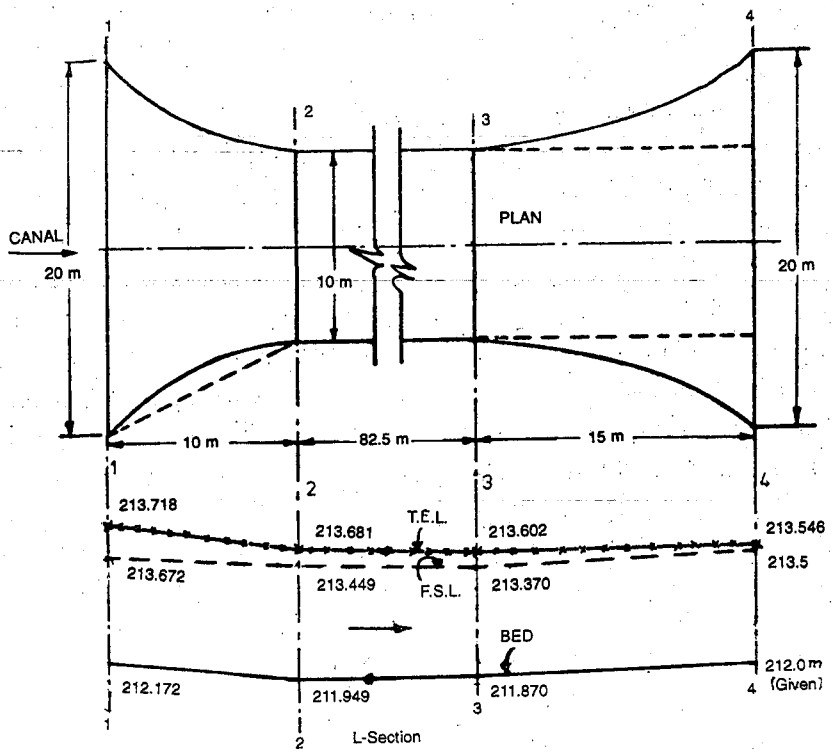


Fig. 14.20. Plan and Section of Canal Trough in Example 14.1.

R.L. of water surface at 4-4 = 212.0 + 1.5 = **213.5 m**

R.L. of T.E.L. at 4.4 = 213.5 + 0.046 = 213.546 m

The known condition of 4-4 shall now be utilised for finding the bed levels etc. at 3.3.

At Section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have at section 3.3 :

Bed width = 10 m

Area of channel = $10 \times 1.5 = 15$ sq m

$$\text{Velocity} = V_3 = \frac{32}{15} = 2.13 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9.81} = 0.232 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left[\frac{V_3^2 - V_4^2}{2g} \right]$$

$$= 0.3 \, [0.232 - 0.046]$$

$$= 0.3 \times 0.186 = 0.0558 \text{ m ; say } \mathbf{0.056 \text{ m}}$$

R.L. of T.E.L. at section 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion
$= 213.546 + 0.056 = 213.602$ m

$\therefore$ R.L. of water surface at 3-3 = R.L. of T.E.L. at 3-3 – Velocity Head
$= 213.602 - 0.232 =$ **213.370 m**

R.L. of bed at 3.3
$= 213.370 - 1.5 =$ **211.87 m**

At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3 which may be computed by Manning's formula as equal to

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}.$$

where n is rugosity coefficient whose value in concrete trough may be taken as 0.016; and L is the length of trough = 82.5 m.

Area of trough section $(A) = 10 \times 1.5 = 15$ sq m

Wetted perimeter $(P) = 10 + 2 \times 1.5 = 13$ m

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{15}{13} = 1.16$ m

Velocity in trough $= \frac{Q}{A} = \frac{32}{15} = 2.13$ m/sec

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{4/3}}$$

$= 0.0787$ m; say **0.079 m**

R.L. of T.E.L. at 2-2 = R.L. of T.E.L. at 3-3 + Friction loss in trough
$= 213.602 + 0.079 = 213.681$ m

R.L. of water surface at 2-2
$= 213.681 - 0.232 =$ **213.449 m**

R.L. of bed at 2-2
$= 213.449 - 1.5 =$ **211.949 m**

At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$$

$$= 0.2\left[\frac{(2.13)^2 - (0.947)^2}{2 \times 9.81}\right]$$

$= 0.2\,[0.232 - 0.046] =$ **0.037 m**

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction
$= 213.681 + 0.037 =$ **213.718 m**

R.L. of water surface at 1-1

$$= 213.718 - 0.046 = \mathbf{213.672\ m}$$

R.L. of bed at 1-1

$$= 213.672 - 1.5 = \mathbf{212.172\ m}$$

All the bed levels, F.S.L. and T.E.L. are plotted in Fig. 14.20.

Step 4. Design of Transitions

(*a*) *Contraction Transition.* Since the depth is kept constant, the transition can be designed on the basis of Mitra's Hyperbolic transition equation (14.2) given as :

$$B_x = \frac{B_n \cdot B_f L_f}{L_f B_n - x (B_n - B_f)}$$

where $B_f = 10$ m

$B_n = 20$ m

$L_f = 10$ m

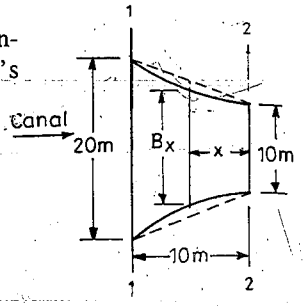


Fig. 14.21

Substituting we get

$$B_x = \frac{20 \times 10 \times 10}{10 \times 20 - x (20 - 10)} = \frac{2{,}000}{200 - 10x}$$

For various values of x lying between 0 to 10 m, various values of B_x are worked out, as shown below in Table 14.2. The distance x is measured from flumed section *i.e.* 2-2, as shown in Fig. 14.21.

Table 14.2

x in metres	0	2	4	6	8	10
$B_x = \frac{2{,}000}{200 - 10x}$ *in metres*	10.0	11.11	12.5	14.29	16.67	20.0

The contraction transition can be plotted with these values.

Expansion Transition. In this case $B_n = 20$ m, $B_f = 10$ m, and $L_f = 15$ m.

Using Eqn. (14.2), we get

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f \cdot B_n - x (B_n - B_f)}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x (20 - 10)} = \frac{3{,}000}{300 - 10x}$$

For various values of x lying between 0 to 15 m, various values of B_x are worked out by using the above equation, as shown in Table 14.3.

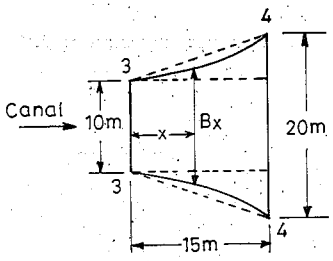
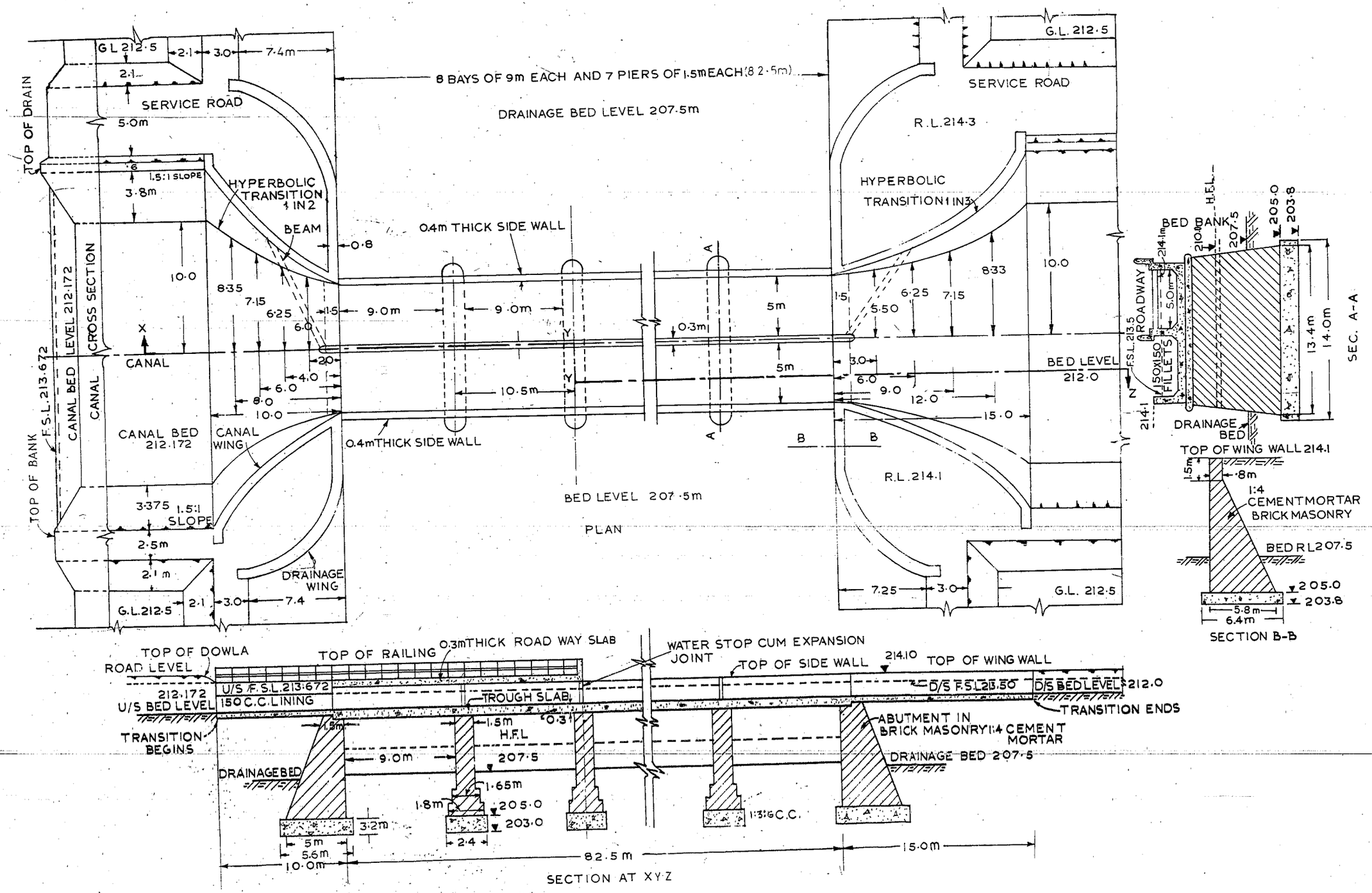


Fig. 14.22

Table 14.3

x in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{3{,}000}{300 - 10x}$ *in metres*	10.0	10.71	11.54	12.5	13.64	15.0	16.67	18.75	20.0

The expansion transition can be easily plotted with these values.



DETAILS OF THE AQUEDUCT (FIG. 14.24.)

Step 5. Design of Trough

The trough shall be divided into two equal compartments of 5 m each and separated by an intermediate wall of 0.3 m thickness. The inspection road shall be carried on the top of left compartment as shown in Fig. 14.23.

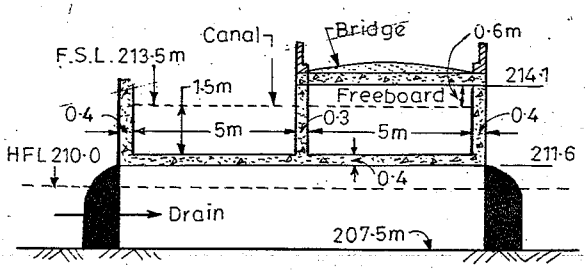


Fig. 14.23

A freeboard of 0.6 m above the normal water depth of 1.5 m is sufficient, and hence, the bottom level of bridge slab over the left compartment can be kept at 1.5 + 0.6 = 2.1 m above the bed level of the trough. The height of the trough will, therefore, be kept equal to 2.1 m. The entire trough section will be constructed in monolithic reinforced concrete and can be designed by usual structural methods. The tentative thicknesses may be used as follows :

Outer walls = 0.4 m thick
Bottom slab of trough = 0.4 m thick

The intermediate partition wall is to be extended in the transitions so as to provide the necessary clear width of 10 m. The detailed drawing of the aqueduct is illustrated in attached chart Fig. 14.24.

Example 14.2. *Design a syphon aqueduct if the following data at the crossing of a canal and a drainage are given :*

(*i*) *Discharge of canal*	= *40 cumecs.*
(*ii*) *Bed width of canal*	= *30 m.*
(*iii*) *Full supply depth of canal*	= *1.6 m.*
(*iv*) *Bed level of canal*	= *206.4 m.*
(*v*) *Side slopes of canal*	= $1\frac{1}{2}$ *H : 1 V.*
(*vi*) *High flood discharge of drainage*	= *450 cumecs*
(*vii*) *High flood level of drainage*	= *207.0 m.*
(*viii*) *Bed level of drainage*	= *204.5 m.*
(*ix*) *General ground level*	= *206.5 m.*

Solution. Since the drainage is of a large size, work of type III will be adopted. Further, because the canal bed level (206.4 m) is slightly below the drainage HFL (207.0 m) ; **a syphon aqueduct** is required and is also asked for. The earthen banks of the canal will be discontinued and the canal water taken in a concrete trough. For affecting economy, the canal shall be flumed.

Step 1. Design of Drainage Waterway

Lacey's regime perimeter $= P = 4.75\sqrt{Q}$

$= 4.75\sqrt{450} = 100.8$ m

Provide 11 clear spans of 8 m each and let the width of each pier be 1.5 m.

The length occupied by 11 bays of 8 m each = $11 \times 8 = 88$ m

The length occupied by 10 piers of 1.5 each = $10 \times 1.5 = 15$ m

Total length of waterway = $88 + 15 = \mathbf{103\ m.}$

Let us, now, limit the velocity through syphon-barrels, to a value, say 2 m/sec.

Height of barrels required

$$= \frac{\text{Discharge}}{\text{Velocity} \times \text{clear width of waterway}} = \frac{450}{2 \times 83} \text{ m} = 2.56 \text{ m}.$$

Hence, provide 11 rectangular barrels, each 8 m wide and 2.5 m high.

Actual velocity through barrels $= \frac{450}{11 \times 8 \times 2.5} = 2.05$ m/sec.

Step 2. Design of Canal Waterway

Normal bed width of canal = 30 m

Let the width be reduced to 15 m.

Providing a splay of 2 : 1 in contraction, the length of contraction transition

$$= \frac{30 - 15}{2} \times 2 = 15 \text{ m}.$$

Providing a splay of 3 : 1 in expansion, the length of expansion transition

$$= \frac{30 - 15}{2} \times 3 = 22.5 \text{ m}.$$

Length of flumed rectangular portion of the canal between abutments = 103 m (provided). In transitions, the side slopes of the canal section shall be warped in plan from the original slope of $1\frac{1}{2}H : 1V$ to vertical.

Step 3. Design of Bed Levels at Different Sections (Fig. 14.25)

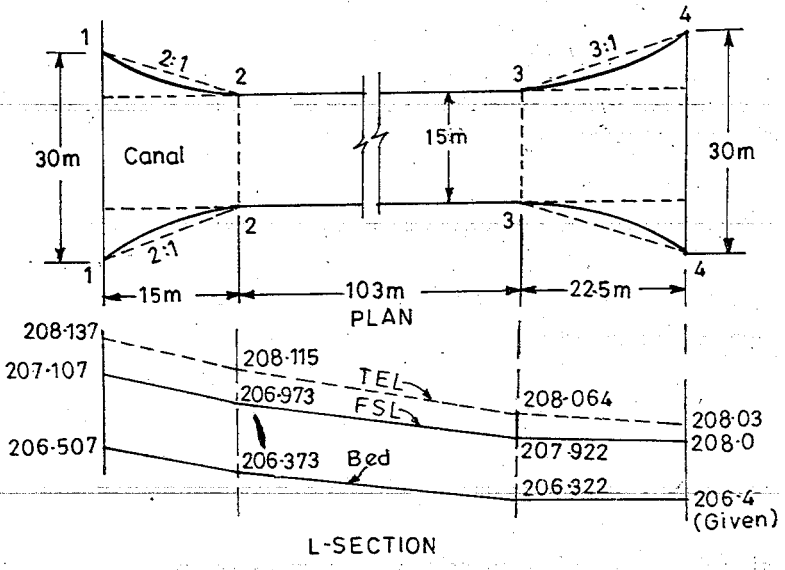


Fig. 14.25. Plan and Section of Canal Trough in Example 14.25.

At Section 4-4

When the canal returns to its normal section, we have the known conditions as follows :

Area of trapezoidal canal section

$= (B + 1.5y)\, y$

where B = Bed width = 30 m.

y = Depth = 1.6 m

$= [30 + 1.5 \times 1.6]\ 1.6$

$= 32.4 \times 1.6 = 51.84$ sq. m.

Velocity of flow $= V_4 = \dfrac{Q}{A} = \dfrac{40}{51.84} = 0.77$ m/sec.

Velocity head $= \dfrac{V_4^2}{2g} = \dfrac{(0.77)^2}{2 \times 9.81} = 0.030$ m

R.L. of canal bed at 4-4 = **206.4 m** (given)

Water depth = 1.6 m (given)

R.L. of water surface at 4-4 = 206.4 + 1.6 = **208.0 m**

R.L. of T.E.L. at 4-4 = 208.0 + 0.03 = 208.03 m.

At Section 3-3

Assuming a constant depth of 1.6 m throughout the channel, we have at section 3-3, a rectangular channel, as follows :

Bed width = 15 m

Depth = 1.6 m (assumed constant)

Area = $15 \times 1.6 = 24$ sq. m.

Velocity $= V_3 = \dfrac{40}{24} = 1.67$ m/sec.

Velocity head $= \dfrac{V_3^2}{2g} = \dfrac{(1.67)^2}{2 \times 9.81} = 0.142$ m

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3\left[\frac{V_3^2 - V_4^2}{2g}\right]$$

$= 0.3\ [0.142 - 0.030] = 0.3 \times 0.112 = 0.0336$ m; **say 0.034 m.**

R.L. of T.E.L. at 3-3 = R.L. of T.E.L. at 4-4 + Loss in expansion

$= 208.030 + 0.034 = 208.064$ m.

R.L. of water surface at 3-3

$= 208.064 - 0.142 =$ **207.922 m.**

R.L. of bed at 3-3 $= 207.922 - 1.6 =$ **206.322 m.**

At Section 2-2

From section 2-2 to section 3-3, the trough section is constant. Therefore, the area and velocity at 2-2 are the same as at 3-3. There is a friction loss between 2-2 and 3-3, which may be computed by Manning's formula, as equal to

$$H_L = \frac{n^2 V^2 L}{R^{4/3}}$$

where n is rugosity coefficient, whose value in a concrete trough may be taken as 0.016 and L is the length of channel = 103 m.

Area of trough section $(A) = 15 \times 1.6 = 24$ sq. m

Wetted perimeter $= 15 + 2 \times 1.6 = 18.2$ m

Hydraulic mean depth $= R = \frac{A}{P} = \frac{24}{18.2} = 1.32$ m

Velocity in trough $= \frac{Q}{A} = \frac{40}{24} = 1.67$ m/sec

Head loss, $H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} = \frac{(0.016)^2 \times (1.67)^2 \times 103}{(1.32)^{4/3}} = \mathbf{0.051\ m.}$

R.L. of T.E.L. at 2-2 = R.L. of T.E.L. at 3-3 + Head loss in trough

$= 208.064 + 0.051 = 208.115$ m.

R.L. of water level at 2-2 $= 208.115 - 0.142 = \mathbf{207.973\ m.}$

R.L. of bed at 2-2 $= 207.973 - 1.6 = \mathbf{206.373\ m.}$

At Section 1.1

Loss of head in contraction transition from section 1-1 to section 2-2 may be taken as

$$= 0.2\left[\frac{V_2^2 - V_1^2}{2g}\right] = 0.2\left[\frac{(1.67)^2 - (0.77)^2}{2 \times 9.81}\right] = 0.0224\ \text{m ; say } \mathbf{0.022\ m.}$$

R.L. of T.E.L. at 1-1 = R.L. of T.E.L. at 2-2 + Loss in contraction

$= 208.115 + 0.022 = 208.137$ m

R.L. of water surface at 1-1

$= 208.137 - 0.030 = \mathbf{208.107\ m.}$

R.L. of bed at 1-1 required to maintain constant depth

$= 208.107 - 1.6 = \mathbf{206.507\ m.}$

All these levels are plotted and shown in Fig. 14.25.

Step 4. Design of Transitions

(*a*) *Contraction Transition.* Since depth is kept constant, the transition shall be designed on the basis of Mitra's Hyperbolic transition equation, (14.2) given by

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)}$$

where $B_f = 15$ m

$B_n = 30$ m

$L_f = 15$ m.

Substituting, we get

$$B_x = \frac{30 \times 15 \times 15}{30 \times 15 - x(30 - 15)} = \frac{6750}{450 - 15x} = \frac{450}{30 - x}$$

For various values of x lying between 0 to 15 m, various values of B_x are worked out, as shown in Table 14.4. The distance x is is measured from the flumed section 2-2.

Table 14.4

x in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{450}{30 - x}$ in metres	15.0	16.04	17.27	18.72	20.42	22.5	25.0	28.1	30.0

Expansion Transition. In this case, we have

$$B_n = 30 \text{ m}$$
$$B_f = 15 \text{ m}$$
$$L_f = 22.5 \text{ m}$$
$$B_x = \frac{B_n \cdot B_n \cdot L_f}{B_n \cdot L_f - x(B_n - B_f)} \quad \textit{i.e.} \text{ Eq. (14.2)}$$
$$= \frac{30 \times 15 \times 22.5}{30 \times 22.5 - x(30 - 15)} = \frac{675}{45 - x}$$

For various values of x lying between 0 to 22.5 m, corresponding values of B_x are worked out, as shown in Table 14.5. The distance x is measured from the flumed section AB.

Table 14.5

x in metres	0	2	4	6	8	10	12	14	16	18	20	22.5
$B_x = \frac{675}{45 - x}$ in metres	15.0	15.7	16.46	17.3	18.25	19.3	20.4	21.75	23.3	25.0	27.0	30.0

Step 5. Design of Trough

The trough shall be divided into three equal compartments, each 5 m wide, separated by 0.3 m thick partition walls (2 Nos.). The inspection road (5 m wide) shall be carried in the extreme left compartment, as shown in Fig. 14.28. A free-board of 0.6 m above normal water depth of 1.6 m is sufficient, and hence, the bottom level of bridge slab may be kept at $1.6 + 0.6 = 2.2$ m above the bed level of the trough. The height of the trough will also be kept as equal to 2.2 m. The entire trough section can be designed as monolithic reinforced concrete structure by the usual structural methods. The tentative thicknesses may be used as follows :

Outer walls = 0.4 m thick

Bottom slab of trough = 0.4 m thick.

The intermediate walls shall be extended into transitions, so as to provide the necessary clear width of 15 m.

Now, the overall outer width of trough (including walls)

$$= 15 + 2 \times 0.3 + 2 \times 0.4$$
$$= 15 + 0.6 + 0.8 = \mathbf{16.4 \text{ m}}.$$

Hence, the length of syphon barrel = **16.4 m**

Step 6. Head Loss Through the Syphon Barrels

The head loss through the syphon barrels is given by Unwin's formula as equal to (neglecting vel. of approach)

$$h = \left[1 + f_1 + f_2 \cdot \frac{L}{R}\right]\frac{V^2}{2g}$$ *i.e.* Eq. (14.1)

where V = velocity through barrels = 2.05 m/sec

f_1 = coefficient of loss of head at entry,

= 0.505 for unshaped mouth.

$f_2 = a\left(1 + \frac{b}{R}\right)$ where the values of a and b are taken from table 14.1 for cement plastered barrels as

$a = 0.00316$

$b = 0.030$

R = Hydraulic mean depth for barrel.

$= \frac{A}{P} = \frac{8 \times 2.5}{2\,(8 + 2.5)} = \frac{20}{21} = 0.953$

L = Length of barrel = 16.4 m.

Substituting these values, we get

$$f_2 = 0.00316\left[1 + \frac{0.030}{0.953}\right] = \mathbf{0.00326}$$

$$\therefore \quad h = \left[1 + 0.505 + 0.00326\left(\frac{16.4}{0.953}\right)\right]\frac{(2.05)^2}{2 \times 9.81} = \mathbf{0.333\ m}$$

High flood level of Drainage is given = 207.0 m

$\therefore$ d/s H.F.L. = 207.0 m

Afflux (h) = 0.333 m

u/s H.F.L. = d/s H.F.L. + Afflux (or loss of head)

= 207.0 + 0.333 = **207.333 m.**

Step 7. Uplift Pressure on Roof of barrels

R.L. of bottom of trough = R.L. of canal bed – Slab thickness

= 206.4 – 0.4 = 206.0 m

Loss of head at entry of barrel $= 0.505\frac{V^2}{2g} = 0.505 \times \frac{(2.05)^2}{2 \times 9.81} = 0.108$ m.

Uplift on the roof

= u/s H.F.L. – Loss at entry – Level of underside of roof slab

= 207.333 – 0.108 – 206.0

= 1.225 m of water = 12.25 kN/m^2 (1.225 t/m^2)

(Assuming unit wt. of water = 10 kN/m^3 or 1 t/

The concrete trough slab is 0.4 m thick and will thus exert a downward load

$0.4 \times 24 = 0.96$ kN/m^2 (assuming unit wt. of concrete = 24 kN

* Strictly speaking, unit wt. of water = 9.81 kN/m^3 ; but to ease in calculations we have taken unit water = 10 kN/m^3 (1 t/m^3) and of concrete = 24 kN/m^3.

The balance of the uplift pressure *i.e.* $12.25 - 9.6 = 2.65\ kN/m^2$ has to be resisted by the reinforcement to be provided at the top in the roof slab. The roof slab has to be designed for full canal water load (1.6 m of water) plus self weight, when the drainage water is low and not exerting any uplift. Suitable reinforcement at bottom of the slab may be provided for this downward force, as worked out below :

Step 8. Design of roof of barrel

Uplift to be balanced by top reinforcement $= 2.65\ kN/m^2$

Downward water load acting when there is no uplift = 1.6 m of water $= 16\ kN/m^2$

Load due to self wt. of slab $= 0.4 \times 24 = 9.6\ kN/m^2$

Total downward load $= 16 + 9.6 = 25.6\ kN/m^2$

An intermediate wall of 0.3 m thickness has been provided in the trough, having clear span of 5 m, the effective span is, therefore, $= 5 + 0.3 = 5.3$ m.

Let us consider 1 m wide strip of slab.

Maximum sagging bending moment in slab due to downward loads

$$= \frac{25.6 \times (5.3)^2}{10}\ kN\text{–}m = 71.7\ kN\text{–}m = 71.7 \times 10^5\ N.cm$$

Maximum Hogging moment due to residual uplift acting from below

$$= \frac{2.65 \times (5.3)^3}{10}\ kN\text{–}m = 7.42\ kN\text{–}m = 7.42 \times 10^5\ N.cm$$

Max. shear force $= \frac{wl}{2} = \frac{25.6 \times 5.0}{2}\ kN = 64\ kN$

Using 1 : 2 : 4 cement concrete, we have

Effective depth (d) of slab required

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{71.7 \times 10^5}{87 \times 100}} = \mathbf{28.8\ cm}$$

Provided overall thickness is 40 cm and thus provided effective depth

$d = 37.5$ cm.

Steel required at the bottom of the slab

$$= \frac{71.7 \times 10^5}{12000 \times 0.87 \times 37.5}\ cm^2/m\ length$$

(using reduced stress in steel as $12000\ N/cm^2$)

$$= 18.3\ cm^2$$

Provide 16 mm ϕ bars @ 10 cm c/c in the bottom of the slab.

Steel required at top

$$= \frac{7.42 \times 10^5}{12000 \times 0.87 \times 37.5} = 1.9\ cm^2\ (Nominal)$$

Provide 10 mm ϕ bars @ 15 cm/cc.

Also provide 10 mm ϕ bars @ 20 cm c/c as distribution reinforcement both at top as well as at bottom, as shown in Fig. 14.26.

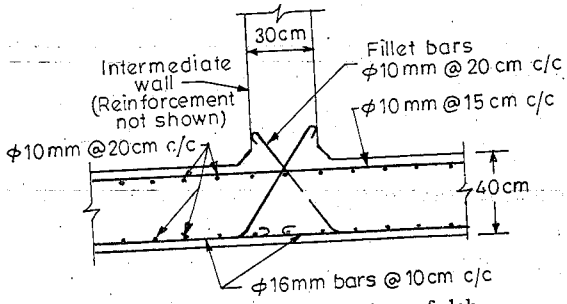


Fig. 14.26. Reinforcement in roof slab.

Step 9. Uplift on the bottom floor of barrel

(*a*) *Static Head*

R.L. of barrel floor = R.L. of trough bottom – Height of barrel
= 206.0 – 2.5 = 203.5 m.

Let us assume that a thickness of 0.8 m is provided.

∴ R.L. of bottom of floor
= 203.5 – 0.8 = 202.7 m

Bed level of drain = 204.5 m.

Assuming that the water-table has gone upto bed level of drain, the static uplift on the floor (refer Fig. 14.18)
= 204.5 – 202.7 = 1.8 m of water

(*b*) *Seepage Head.* The seepage head will be maximum when the canal is running full and the drain is dry.

Thus, the total seepage head = F.S.L. of canal – bed level of drain
= 208.0 – 204.5 = 3.5 m.

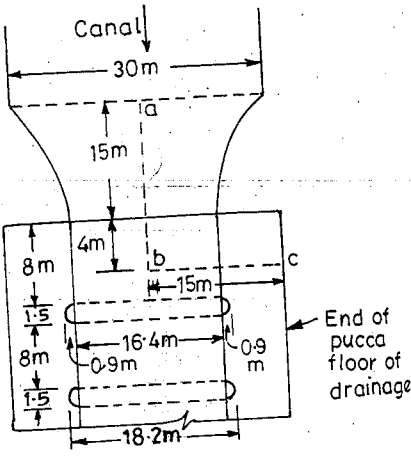


Fig. 14.27

The residual seepage head at a point '*a*' in the centre of the first barrel (Fig. 14.27) has been calculated by Bligh's theory, as follows :

Assuming that the total length of drainage floor = 30 m.

The seepage line *abc* will traverse creep lengths as follows :

ab = Length of *u/s* transition + Half the barrel span = 15 + 4 = 19 m.

bc = 15 m (Half of the total length of 30 m = assumed)

Total creep length = 19 + 15 = 34 m.

$$\text{Residual seepage head at } b = 3.5\left[1 - \frac{19}{34}\right] = 3.5 \times \frac{15}{34} = 1.55 \text{ m.}$$

Total uplift = Static head + Seepage head
= 1.8 + 1.55 = 3.35 m of water = 33.5 kN/m^2

The provided 0.8 m thickness of slab will resist due to its own wt., an uplift

$$= 0.8 \times 24 = 19.2 \text{ kN/m}^2.$$

$\therefore$ Balance to be resisted by reinforcement due to bending action

$$= 33.5 - 19.2 = 14.3 \text{ kN/m}^2.$$

Suitable reinforcement for this uplift (*i.e.* 14.3 kN/m^2) has to be provided at the top of the culvert floor so as to counteract the bending action.

Note. The length of the floor has been provided equal to 32 m, as shown in Fig. 14.28, on the following considerations.

(*i*) Length of floor required under barrel	= 16.4 m
(*ii*) Extra floor length required to accommodate pier noses on both sides = 2 × 1.0	= 2.0 m
(*iii*) Horizontal length of d/s ramp joining to bed level at a slope of 5 : 1 = 5 (204.5 – 203.5)	= 5.0 m
(*iv*) Width of d/s cut-off beyond ramp	= 0.6 m
(*v*) Length of extra floor provided on u/s side	= 0.6 m
	Total length = 30.0 m

Step 10. Design of cutoffs and protection works for the drainage floor

The depth of scour $(R) = 0.47\left[\frac{Q}{f}\right]^{1/3}$; assuming $f = 1.0$

$$\therefore \quad R = 0.47\left(\frac{450}{1}\right)^{1/3} = 0.47 \times 7.65 = 3.59 \text{ m}$$

Provide depth of cut-offs for scour hole of 1.5 R on both sides

Depth of u/s cut off below H.F.L.

$$= 1.5R = 1.5 \times 3.59 = 5.4 \text{ m}$$

R.L. of bottom of u/s cut-off = u/s H.F.L. – 5.4 m

$$= 207.333 - 5.4 = 201.933 \text{ m ; say } \mathbf{201.93 \text{ m.}}$$

R.L. of bottom of d/s cut-off = d/s H.F.L. – 5.4

$$= 207.0 - 5.4 = \mathbf{201.6 \text{ m}}$$

Length of u/s protection (*i.e.* 40 cm thick brick pitching)

$$= 2 \text{ [R.L. of u/s bed} - \text{R.L. of bottom of u/s cut-off]}$$

$$= 2\,[203.50 - 201.93]$$

$$= 2 \times 1.57 = 3.14 \text{ m ; say } \mathbf{3.2 \text{ m}}$$

Similarly, length of d/s brick pitching

$$= 2 \text{ [R.L. of d/s bed} - \text{R.L. of bottom of d/s cutoff]}$$

$$= 2\,[204.5 - 201.6] = 2 \times 2.9 = \mathbf{5.8 \text{ m}}$$

The pitchings may be supported by 0.4 m wide and 1 m deep toe walls, as shown in Fig. 14.28.

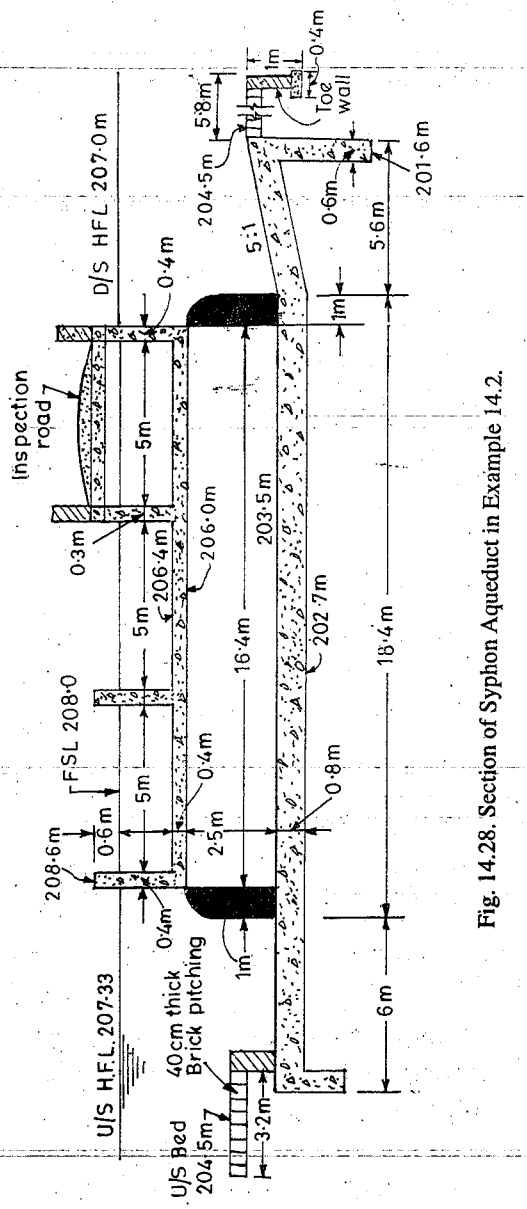


Fig. 14.28. Section of Syphon Aqueduct in Example 14.2.

Example 14.3. *Design a suitable cross drainage work, given the following data at the site of the crossing of two streams of water :*

(a) Irrigation channel :

Full supply discharge = *350 cumecs*

Full supply level = *202.5 m.*
Canal bed level = *197.8 m.*
Canal bed width = *35 m.*
Full supply depth = *4.7 m.*
Side slopes = $\frac{1}{2}H : IV$

(b) Natural Drainage :
Drainage bed level = *203.9 m.*
High flood level = *205.2 m.*
Catchment area of drainage up to crossing = *14.3 sq. km.*

The Dicken's formula may be used for computing H.F.Q. with its coefficient as 18 (SI or MKS units).

Solution. The high flood discharge of the drainage at the point of the crossing may be obtained by using Dicken's formula, which states

$$Q = C \cdot A^{3/4} = 18 \cdot (14.3)^{3/4} = 18 \times 7.33 = \mathbf{132\ cumecs}.$$

Since the bed level of drainage (203.9 m) is much above the canal *FSL* (202.5 m), *i.e.* by about 1.4 m ; the canal water will be taken below the drainage. Hence, the cross-drainage work to be constructed at the crossing will be a **super-passage**. The design of the super passage is to be done on the same lines as that of an aqueduct, and is given below :

Step 1. Design of Canal Water-way

The flow velocity in the canal (as it is)

$$\frac{\text{Discharge}}{\text{Area}} = \frac{Q}{(B + \frac{1}{2}y)\, y} \text{ (using } \tfrac{1}{2}H : 1V)$$

$$= \frac{350}{(35 + 0.5 \times 4.7)\, 4.7} = \frac{350}{37.35 \times 4.7} = 1.99 \text{ m/sec.}$$

This high velocity shows that the canal is already a lined canal, and much more fluming can not be affected. Hence, the original bed width of 35 m can be continued as canal barrels below the drainage trough, or slight fluming may be done. *Let us adopt a clear waterway of 30 m, in two spans, each of 15 m,* with a central pier of width say 1.5 m, thus providing an overall linear waterway of 31.5 m between abutments, and this will be the length of drainage troughs.

Providing a splay of 2 : 1 in contraction (on u/s side), the length of contraction transition

$$= \frac{35 - 31.5}{2} \times 2 = 5 \text{ m}$$

Providing a splay of 3 : 1 in d/s expansion, the length of expansion transition

$$= \frac{35 - 31.5}{2} \times 3 = 7.5 \text{ m}$$

Length of flumed rectangular portion of canal will be equal to the width of the drainage troughs, as worked out in the next step (*i.e.* 50.5 m).

The piers, abutments, wing walls and return walls of the canal will be designed as those of a bridge, taking the load of drainage trough (including the load of water and inspection road, etc.), instead of a bridge deck slab.

Step II. Design of Drainage Waterway

Lacey's regime perimeter $P = 4.75\sqrt{Q} = 4.75\sqrt{132} = 54.5$ m.

The total length of the waterway provided is generally chosen equal to P, although it can be slightly reduced to affect economy, but too much contraction of the drainage poses problems, and hence too much fluming is never done.

Let us provide 6 R.C.C. compartments, each of clear width equal to 8.0 m, thus giving

Clear waterway $= 8.0 \times 6 = 48$ m.

Using 5 partition walls of 0.3 m thick each,

length occupied by walls $= 5 \times 0.3 = 1.5$ m

$\therefore$ Total length of waterway provided $= 48 + 1.5 = \mathbf{49.5\ m}$; against P of 54.5 m (which is not too less as to cause troubles)

The two side walls of the R.C.C. drainage trough may be kept 0.4 m thick each, with 49.5 m as aggregate waterway between them. Thus, end to end length of drainage trough $= 49.5 + 0.8 = 50.5$ m.

Thus, the length of the rectangular portion of canal will also be equal to 50.5 m.

Since the drainage has also been slightly flumed and kept lesser than P, so let us design its contraction and expansion lengths.

Assuming 2 : 1 convergence, we have the length of contraction transition

$$= \frac{54.5 - 49.5}{2} \times 2 = 5 \text{ m}$$

Assuming 3 : 1 splay in expansion, we have the length of expansion transition

$$= \frac{54.5 - 49.5}{2} \times 3 = 7.5 \text{ m}$$

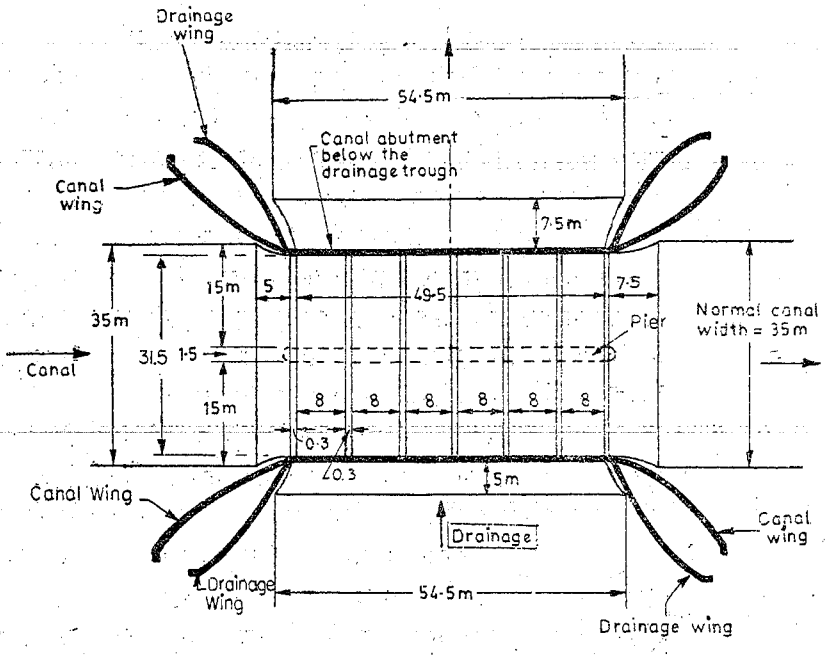


Fig. 14.29. Indicative Plan of the Super-passage Crossing in Example 14.3.

The wing walls will be constructed to reduce the drainage waterway width from 54.5 m to 49.5 m on upstream, and return walls will be constructed to expand the drainage waterway from 49.5 m to 54.5 m on downstream. These wings will be extended so as to enter the berms of the drain.

The length of the drainage pucca rectangular trough will be equal to 31.5 m (*i.e.* equal to the rectangular waterway of canal). A line plan of the crossing is shown in Fig. 14.29.

Step 3. Head Loss and Bed Levels at different Sections along the length of drainage trough.

At section 4-4 (Refer Fig. 14.30)

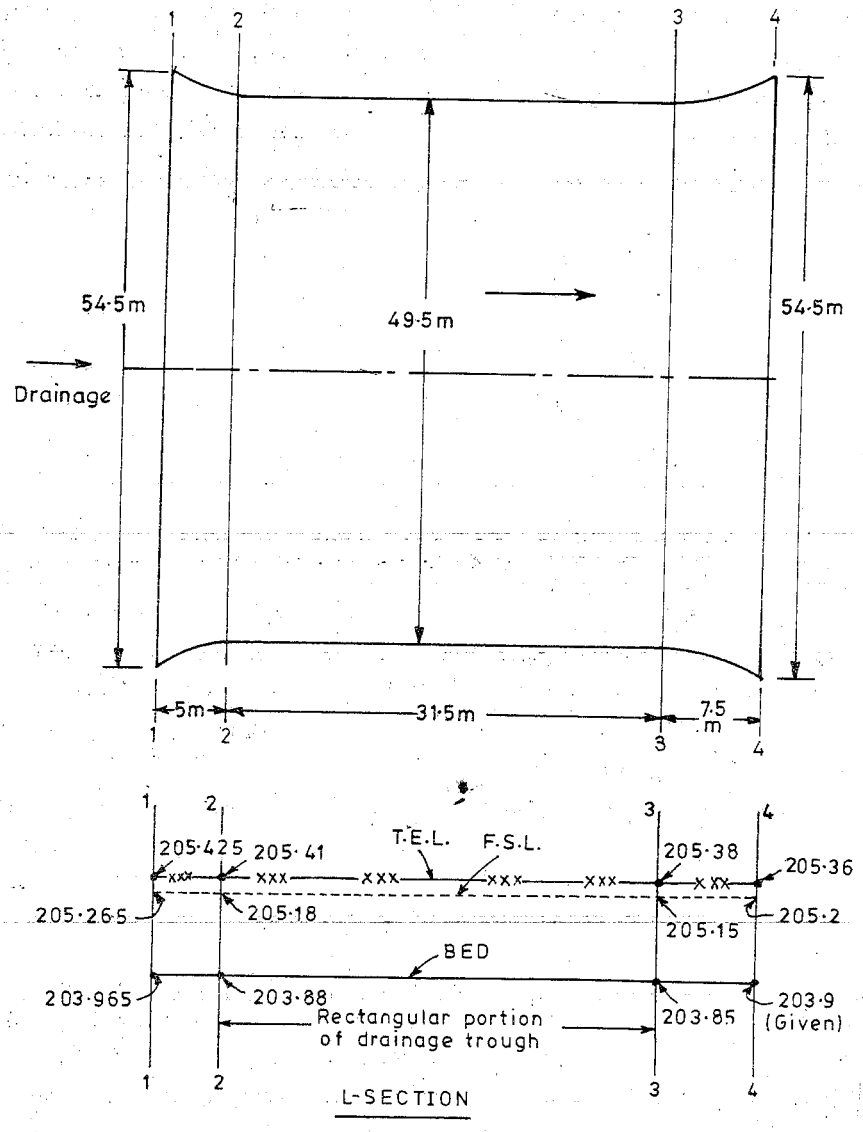


Fig. 14.30. Plan and L-section of Drainage trough carried over the canal.

At section 4-4, where the drainage returns to its normal section, we have the area of the natural drainage section

$$= \text{width} (\approx \text{perimeter}) \times \text{Depth}$$
$$= 54.5 \times (205.2 - 203.9) = 54.5 \times 1.3 = 70.85 \text{ m}^2$$

$$\text{Velocity} = V_4 = \frac{Q}{A} = \frac{132}{70.85} = 1.86 \text{ m/sec.; say } \mathbf{1.9 \text{ m/sec.}}$$

$$\text{Velocity Head} = \frac{V_4^2}{2g} = \frac{(1.9)^2}{2 \times 9.81} = 0.16 \text{ m}$$

R.L. of bed at 4-4 = **203.9 m** (given)

R.L. of water surface at 4-4

$$= 203.9 + 1.3 \text{ (as the normal depth in drainage} = 1.3 \text{ m)}$$
$$= 205.2 \text{ m}$$

R.L. of *TEL* at 4-4 $= 205.2 + 0.16 = 205.36$ m

At Section 3-3 (Refer Fig. 14.30)

Keeping the same depth of 1.3 m throughout the drainage, we have at section 3-3.

Clear waterway $= 8 \times 6 = 48$ m

Area of flow $= 48 \times 1.3 = 62.4 \text{ m}^2$

$$\text{Velocity} = \frac{132}{62.4} = 2.12 \text{ m/sec.}$$

$$\text{Velocity Head} = \frac{(2.12)^2}{2 \times 9.81} = 0.23 \text{ m.}$$

Assuming that the loss of head in expansion from section 3-3 to section 4-4 is taken as

$$= 0.3 \left(\frac{V_3^2 - V_4^2}{2g} \right)$$
$$= 0.3 \ (0.23 - 0.16) = 0.3 \times 0.07 = 0.021 \text{ m; say } \mathbf{0.02 \text{ m}}$$

$\therefore$ R.L. of TEL at 3-3

$$= \text{R.L. of TEL at section 4-4 + Loss in expansion}$$
$$= 205.36 + 0.02 = 205.38 \text{ m}$$

$\therefore$ R.L. of water surface at 3-3

$$= \text{R.L. of TEL at 3-3} - \text{Velocity head}$$
$$= 205.38 - 0.23 = \mathbf{205.15 \text{ m}}$$

$\therefore$ R.L. of bed at 3-3 $= 205.15 - 1.3 = \mathbf{203.85 \text{ m}}$

At Section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are the same as at 3-3. But from 2-2 to 3-3, there is a friction loss between 2-2 and 3-3, which may be computed by Mannings formula, as :

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$$

where n is the rugosity coefficient whose value in concrete trough may be taken as 0.016;

and L is the length of trough = 31.5 m.

The area of trough section $(A) = 48 \times 1.3 = 62.4$ sq m

Wetted perimeter $(P) = 48 + 2 \times 1.3 = 50.6$ m

Hydraulic mean depth $(R) = \frac{A}{P} = \frac{62.4}{50.6} = 1.23$ m

Velocity in trough $= \frac{Q}{A} = \frac{132}{62.4} = 2.12$ m/sec

$$\therefore \quad H_L = \frac{(0.016)^2 \times (2.12)^2 \times 31.5}{(1.23)^{4/3}} = 0.0275 \text{ m ; say } \mathbf{0.03 \text{ m}}$$

R.L. of TEL at 2-2 = R.L. of TEL at 3-3 + Friction loss (H_L) in trough

$= 205.38 + 0.03 = 205.41$ m

R.L. of water surface at 2-2

$= 205.41 - 0.23 = 205.18$ m

∴ R.L. of bed at 2-2

$= 205.18 \text{ m} - 1.3 = \mathbf{203.88 \text{ m}}$

At Section 1-1

Loss of head in contraction transition from 1-1 to 2-2

$$= 0.2\left(\frac{V_2^2 - V_1^2}{2g}\right) = 0.2\,[0.23 - 0.16] = \mathbf{0.014 \text{ m; say } 0.015 \text{ m}}$$

R.L. of TEL at 1-1 = R.L. of TEL at 2-2 + Loss in contraction

$= 205.41 + 0.015 = 205.425$ m

R.L. of water surface at 1-1 $= 205.425 - 0.16 = 205.265$ m

R.L. of bed at 1-1 $= 205.265 - 1.3 = \mathbf{203.965 \text{ m}}$

All these bed levels, FSL and TEL are plotted in Fig. 14.30.

Step 4. Design of Transitions for the Drainage

(*a*) *Contraction Transition.* Since the depth is kept constant, the transitions can be designed on the basis of Mitra's hyperbolic transition equation given by *eqn.* (14.2) as:

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f B_n - x\,(B_n - B_f)}$$

where $B_f = 49.5$ m

$B_n = 54.5$ m

$L_f = 5$ m.

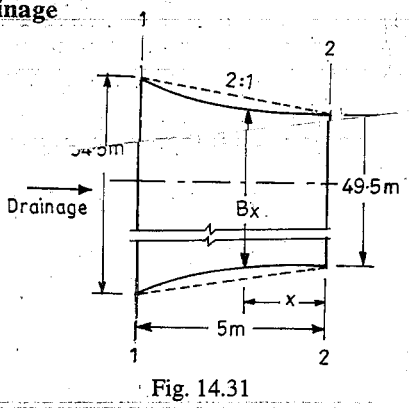


Fig. 14.31

Substituting, we get

B_x = width at any distance x from the flumed section 2–2

$$= \frac{54.5 \times 49.5 \times 5}{5 \times 54.5 - x\,(54.5 - 49.5)}$$

$$= \frac{2700}{54.5 - x}$$

For various values of x lying between 0 to 5 m, values of B are worked out as shown in Table 14.6.

Table 14.6

x in metres	0	1	2	3	4	5
$B_x = \frac{2700}{54.5 - x}$ in metres	49.5	50.5	51.5	52.5	53.5	54.5

The contraction transition can be plotted with these values.

Expansion Transition. In this case $B_x = 54.5$ m, $B_f = 49.5$ m, $L_f = 7.5$ m.

Using Eq. (14.2), we get

$$B_x = \frac{B_n \cdot B_f \cdot L_f}{L_f \cdot B_n - x\,(B_n - B_f)}$$

$$= \frac{154.5 \times 49.5 \times 7.5}{7.5 \times 54.5 - x\,(54.5 - 49.5)}$$

$$= \frac{4050}{81.7 - x}$$

For various values of x lying between 0 to 7.5 m, various values of B_x are worked out by using the above equation, as shown in Table 14.7.

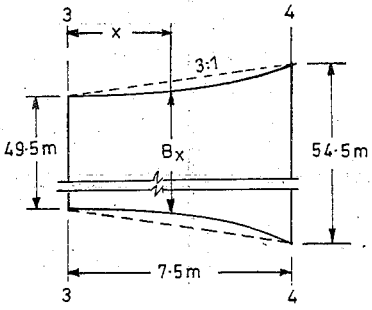


Fig. 14.32

Table 14.7

x in metres	0	2	4	6	7.5
$B_x = \frac{4050}{81.7 - x}$ in metres	49.5	50.7	52.0	53.3	54.5

The expansion transition can be plotted with these values.

Step 5. Design of Drainage Trough

The RCC drainage trough, as pointed out earlier, has been divided into six compartments of 8 m clear width each, and separated by intermediate walls (5 No.) each 0.3 m thick (tentative thickness). The end walls of the trough have tentatively been kept as 0.4 m wide each, as shown in Fig. 14.33. An inspection road may be carried on the top of the end compartment as shown.

A free-board of 0.6 m above the normal water depth of 1.3 m is sufficient, and hence, the bottom-level of bridge slab over the end compartment may be kept $1.3 + 0.6 = 1.9$ m above the bed level of trough (Different bed levels along the length of trough are shown in Fig. 14.30.). The height of the trough walls will also be kept equal to 1.9 m. The entire trough section will be constructed in monolithic reinforced concrete, and can be designed by usual structural methods. The tentative thickness for the walls have already been assumed and should be verified. A detailed drawing for the crossing can be prepared on the same lines on which the drawing 14.24 was prepared.

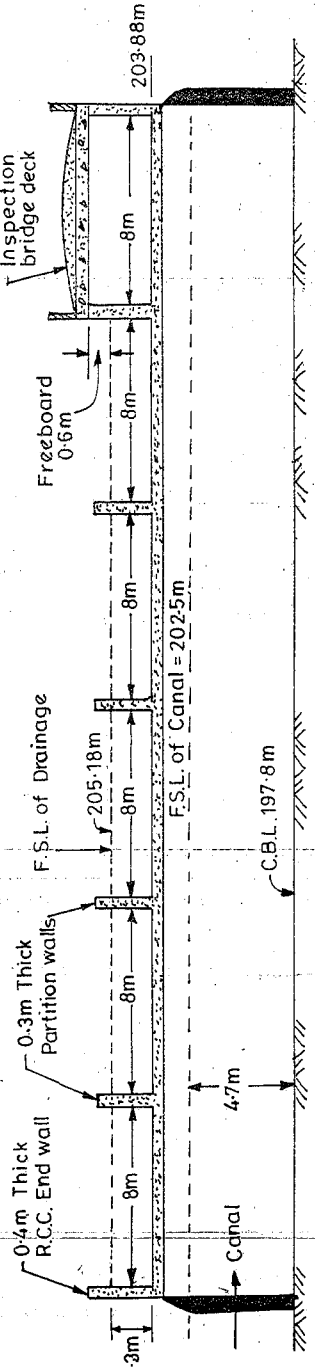


Fig. 14.33. Section of Drainage Trough passing over the Canal
(Levels of Trough bed and FSL at the start of rectangular portion of Trough are shown).

Example 14.4. *Design a suitable cross drainage work, given the following data :*

(a) Irrigation channel

F.S.Q.	*= 354 cumecs.*
Bed width	*= 24 m.*
FSL	*= 207.60 m.*
C.B.L.	*= 201.4 m.*
Side slopes	*= $\frac{1}{2}H : IV.$*

(b) Natural drainage

H.F.Q.	*= 600 cumecs.*
Drainage bed level	*= 203.6 m.*
H.F.L.	*= 206.3 m.*

Solution. Since the bed of the natural drainage (203.6 m) is higher than the canal bed (201.4 m) by a sufficient margin of 2.2 m, we may think of taking the canal below the drainage. But since the FSL of canal (207.6 m), is much above the drainage bed (203.6 m), the simple super passage cannot be provided. The canal water will, therefore, have to be syphoned below the drainage. Since the discharge of the canal is low compared to that of the drainage, such a crossing will not prove uneconomical also. Hence, a **canal syphon** will be designed as below :

Step I. Design of Canal Waterway

Let the velocity through the canal barrels be limited to 3 m/sec.

$\therefore$ Area of waterway of barrels required $= \frac{354}{3} = 118 \text{ m}^2$

Let us provide 4 spans each having a clear waterway of 5.4 m, thus giving a clear waterway of $4 \times 5.4 = 21.6$ m. Let R.C.C. piers (three No.) each of width 0.6 m are provided ; thus occupying a width $3 \times 0.6 = 1.8$ m.

Total overall waterway provided between abutments

$= 21.6 + 1.8$

$= 23.4$ m; which is O.K., as it is less than 24 m (*i.e.* the width of canal)

Assuming 0.3 m as the width of each bearing on abutments, the total length of drainage trough $= 23.4 + 0.6 =$ **24 m.**

Thus, the canal on the u/s and d/s sides is joined to the barrels aggregate width of 23.4 m by suitable transitions and approaches.

$$\text{The height of the barrel} = \frac{\text{Discharge}}{\text{Velocity} \times \text{Waterway width}}$$

$$= \frac{354}{3 \times 21.6} = 5.47 \text{ m.}$$

So let us keep 5.5 m height of the barrels, thus maintaining an actual velocity through them $= \frac{354}{5.5 \times 21.6} = 2.98$ m/sec.

Step 2. Design of Drainage Waterway

Lacey's regime perimeter $= P = 4.75\sqrt{Q} = 4.75 \times \sqrt{600} = 116$ m.

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Let us assume that the rectangular R.C.C. drainage trough is provided in such width that its perimeter becomes equal to that required by Lacey's equation, thus givin width of the trough = 116 – 2 (2.7) = 110.6 m ; say 111 m.

Let us divide the drainage trough into 9 (compartments each of 12 m width (clea width) with 0.4 m thick partition walls (8 No.). Thus the overall width of the troug provided will be equal to $9 \times 12 + 8 \times 0.4 = 108.0 + 3.2 = 111.2$ m. Two end walls eac of 0.4 m thickness can be provided, thus giving the total width of drainage trough en to end = 112 m ; and this will be equal to the length of the canal barrels.

Step 3. Design of Bed Levels along the Drainage Trough

Since the drainage waterway has not been flumed, the only loss of head in TE when the water flows in the trough is due to frictional loss. Assuming that the drainag levels (given) are on the d/s of the crossing.

The bed level of drainage trough (d/s) = 203.6 m.

Velocity in trough, $V = \dfrac{600}{7\,(15.5 \times 2.7)} = 2.05$ m/sec.

Head loss due to friction $(H_L) = \dfrac{n^2 \cdot V^2 L}{R^{4/3}}$

where $R = \dfrac{A}{P} = \dfrac{7\,(15.5 \times 2.7)}{7\,(15.5 + 5.4)} = 2.0$

$$\therefore \qquad H_L = \frac{(0.016)^2 \times (2.05)^2 \times 24}{(2)^{4/3}} = 0.0102 \text{ m.}$$

Hence, the trough should be given a slope (S_0) [as to pass the full drainage d charge] given by

0.0102 m loss in 24 m length

1 m loss will occur in a length $= \dfrac{24}{0.0102}$ m $= 2340$ m.

Hence, the slope required to be given to trough is 1 in 2340 ; and bed level at t d/s of crossing = 203.6 m; and bed level at u/s of crossing = 203.61 m.

Step 4. Design of Transitions

As no fluming either of canal or of drainage has been done, the question designing transitions does not arise. The wing walls and return walls shall, however, being provided, as shown in the attached chart Fig. 14.36, after properly ensuring their str tural safety.

Step 5. Design of Trough

The trough shall be made of reinforced cement concrete and shall be designed by the usual structural methods. A bridge for inspection can be provided on one e end compartment, as shown in Fig. 14.36. The trough is divided into 9 compartments each of 12 m clear width as already decided. The tentative thicknesses of partition walls (0.4 m) and end walls (0.4 m) have also been made.

Step 6. Head Loss Trough Syphon Barrels

The head loss trough syphon barrels is given by Eq. (14.1) as

$$h = \left(1 + f_1 + f_2 \cdot \frac{L}{R}\right) \frac{V^2}{2g} \text{ (neglecting velocity of approach)}$$

where V is the velocity through the syphon barrels
$= 2.98$ m/sec.
f_1 = coeff. of loss of head at entry
$= 0.505$ for unshaped mouth
$f_2 = a\left[1 + \frac{b}{R}\right]$ where the values of a and b are taken from Table 14.1 for cement plastered barrels as
$a = 0.00316$
$b = 0.030$
R = Hydraulic mean depth for barrel
$= \frac{A}{P} = \frac{5.4 \times 5.5}{2\,(5.4 + 5.5)} = \frac{5.4 \times 5.5}{21.8} = 1.36$ m
L = Length of barrels = 112 m.

Substituting these values, we get

$$f_2 = 0.00316\left[1 + \frac{0.030}{1.36}\right] = 0.00323$$

$$\therefore \quad h = \left[1 + 0.505 + 0.00323\left(\frac{112}{1.36}\right)\right]\frac{(2.98)^2}{2 \times 9.81} = 0.855; \text{ say } 0.86 \text{ m.}$$

FSL of canal (given) = 207.6 m
∴ d/s FSL = 207.6 m
Afflux (h) = 0.86 m.
∴ u/s FSL = d/s FSL + Afflux = 207.6 + 0.86 = **208.46 m**

Step 7. Uplift Pressure on Roof of Barrels

R.L. of bottom of trough
= R.L. of drainage bed – Slab thickness
= 203.6 – 0.8 (assuming 0.8 m thick roof slab)
= 202.8 m

Loss of head at entry of barrel $= 0.505\frac{V^2}{2g} = \frac{0.505 \times (2.98)^2}{2 \times 9.81} = 0.23$ m

Uplift on the roof = u/s HFL – Loss at entry – Level of under-side of roof slab
= 208.46 – 0.23 – 202.8
= 5.43 m of water head = 54.3 kN/m^2.

The concrete trough slab is 0.8 m thick and will thus exert a downward load of $0.8 \times 24 = 19.2$ kN/m^2.

The balance of the uplift pressure, *i.e.* 54.3 – 19.2 = 35.1 kN/m^2 has to be resisted reinforcement to be provided at the top in the roof slab. The roof slab has to be igned for full drainage water load (2.7 m of water) plus self-weight, when the canal er is low and not exerting any uplift. Suitable reinforcement, may be provided for downward force, as worked out below :

Step 8. Design of roof barrel

Uplift to be balanced by top reinforcement $= 35.1\ \text{kN/m}^2$

Downward water load acting when there is no uplift $= 2.7\ \text{m of water} = 27\ \text{kN/m}^2$

Load due to self-weight of slab $= 0.8 \times 24 = 19.2\ \text{kN/m}^2$

Total downward load (when there is no uplift) $= 27 + 19.2 = 46.2\ \text{kN/m}^2$

Walls of 0.4 m thickness have been provided in the trough having clear span of 12 m ; the effective span between any two walls is, therefore $= 12 + 0.4 = 12.4$ m

Let us consider 1 m wide strip of slab.

Maximum sagging bending moment in slab due to downward loads (when there is no uplift)

$$= \frac{46.2\,(12.4)^2}{10}\ \text{kN.m.}\ \left(i.e.\ \frac{wl^2}{10}\right) = 708\ \text{kN-m} = 708 \times 10^5\ \text{N.cm}$$

Maximum hogging bending moment due to residual uplift acting from below

$$= \frac{35.1\,(12.4)^2}{10}\ \text{kN} \cdot \text{m} = 538\ \text{kN} \cdot \text{m} = 538 \times 10^5\ \text{N} \cdot \text{cm}$$

$$\text{Max. shear force} = \frac{wl}{2} = \frac{46.2 \times 12.4}{2}\ \text{kN} = 287\ \text{kN}$$

Using 1 : 2 : 4 cement concrete, we have

Economical Effective depth (d) of slab required

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{708 \times 10^5}{87 \times 100}}\ \text{cm} = 90\ \text{cm.}$$

Provided overall thickness is 80 cm and may be kept the same. Thus, effective depth provided, $d = 77.5$ cm.

Steel required at the bottom of the slab

$$= \frac{708 \times 10^5}{12000 \times 0.87 \times 77.5}\ \text{cm}^2/\text{m length}$$

(using reduced stress in steel as 12000 kN/cm^2)

$= 87.8\ \text{cm}^2/\text{m}$ length of slab.

Provide 32 mm dia bars @ 9 cm c/c in the bottom of the slab.

Steel required at top

$$= \frac{538 \times 10^5}{1200 \times 0.87 \times 77.5} = 66.7\ \text{cm}^2/\text{m length of slab.}$$

Provide 32 mm dia bars @ 12 cm c/c.

Also provide 0.15% of gross cross-sectional area of concrete as distribution steel.

$\therefore$ Area of Dist. steel required

$$= \frac{0.15 \times 80 \times 100}{100}\ \text{cm}^2 = 12\ \text{cm}^2$$

$\therefore$ Spacing of 12 mm ϕ bars required

$$= \frac{\text{Area of one bar} \times 100}{\text{Area required}}$$

$$= \frac{1.13 \times 100}{12} = 9.4 \text{ cm; say } 10 \text{ cm c/c}$$

Hence, use 12 mm ϕ bars @ 10 cm c/c as distribution steel, both at top as well as bottom, as shown in Fig. 14.34.

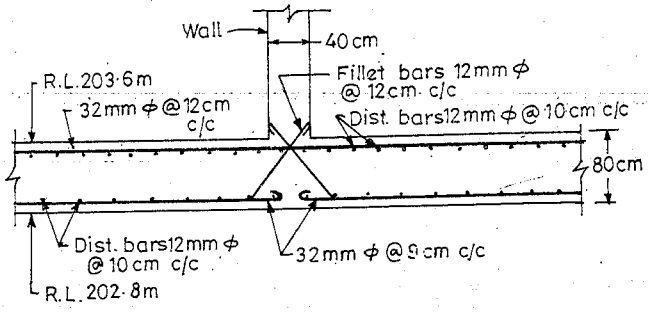


Fig. 14.34. Reinforcement in roof slab.

The partition walls will be designed as beams carrying the bending load from the roof slab.

Step 9. Uplift on the bottom floor of syphon barrels

(*a*) *Static Head*

R.L. of barrel floor = R.L. of trough bottom – Height of barrel

$$= 202.8 - 5.5 = 197.3 \text{ m}$$

Let us assume that a thickness of 0.8 m is provided.

$\therefore$ R.L. of bottom of floor = 197.3 – 0.8 = 196.5 m

Bed level of canal = 201.4 m

Assuming that the water-table has gone upto bed level of the canal, the static uplift on the floor (refer Fig. 14.18)

$$= 201.4 - 196.5 = \mathbf{4.9 \text{ m of water}}$$

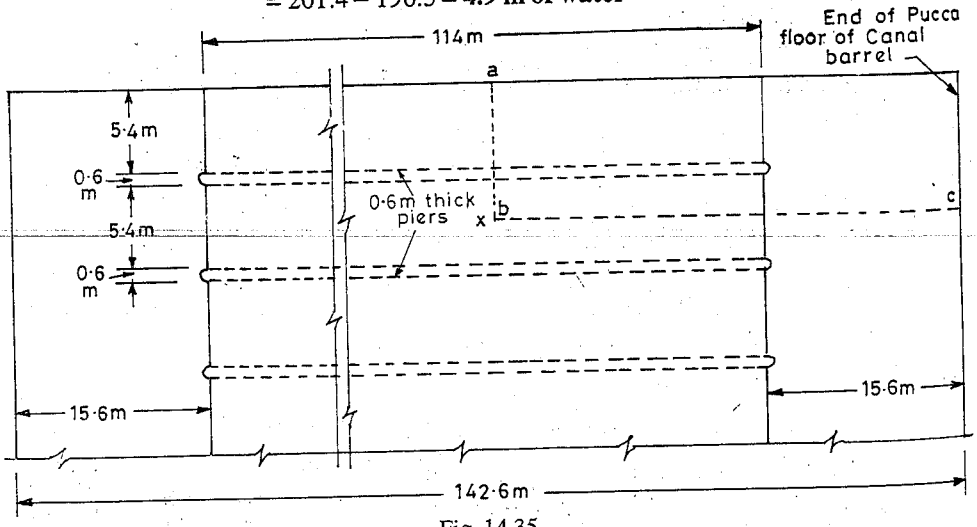


Fig. 14.35

(*b*) *Seepage Head.* The seepage head will be maximum when the drain trough is running full and the canal barrels are dry.

Thus, the total seepage head = H.F.L. of drain – Bed level of canal

$$= 206.3 - 201.4 = 4.9 \text{ m.}$$

The residual seepage head at a point '*a*' in the centre of the first barrel (Fig. 14.35) has been calculated by Bligh's theory as follows :

Assuming that the total length of drainage floor = 142.6 m

The seepage line of drain water *abc* will traverse creep lengths as follows :

$$ab = 5.4 + 0.6 + 2.7 = 8.7 \text{ m} ; \; bc = \frac{142.6}{2} = 71.3 \text{ m}$$

Total creep length = 8.7 + 71.3 = 80 m

$$\text{Residual seepage head at } b = 4.9\left[1 - \frac{8.7}{80}\right] = 4.9 \times \frac{71.3}{80} = 4.37 \text{ m}$$

Total uplift = Static head + Seepage head = 4.9 + 4.37 = 9.27 m = 92.7 kN/m^2

The provided 0.8 m thickness will resist due to its own weight, an uplift

$$= 0.8 \times 24 = 19.2 \text{ kN/m}^2$$

$\therefore$ Balance to be resisted by reinforcement due to bending action

$$= 92.7 - 19.2 = 73.5 \text{ kN/m}^2$$

Max. hogging B.M. in bottom slab

$$= \frac{73.5 \times (5.4 + 0.6)^2}{10}$$

$$= \frac{73.5 \times 6^2}{10} = 264 \text{ kN} \cdot \text{m} = 264 \times 10^5 \text{ N–cm}$$

$$\text{Steel required} = \frac{264 \times 10^5}{12000 \times 8.7 \times 77.5} \text{ cm}^2/\text{m length} = 32.7 \text{ cm}^2$$

Use 32 mm ϕ bars @ 24 cm c/c at top in the barrel floor (*i.e.* bottom floor). 12 mm ϕ bars @ 10 cm c/c both ways at bottom, and one way also at top, may be provided as distribution bars.

The hydraulic dimensions and detailings are shown in attached chart Fig. 14.36.

Step 10. Design of Cutoff's and protection works for the canal floor

$$\text{Lacey's normal depth of scour} = R = 0.47\left(\frac{Q}{f}\right)^{1/3} ; \text{ assuming } f = 1$$

$$R = 0.47 \times \left(\frac{354}{1}\right)^{1/3} = 7 \text{ m below FSL}$$

Provide depth of cut off's for scour holes of 1.5 R on both sides

or $1.5R = 1.5 \times 7 = 10.5$ m below FSL

$\therefore$ RL of bottom of u/s cutoff = 208.46 – 10.5 = **197.96 m**

R.L. of bottom of d/s cut-off = 207.6 – 10.5 = **197.1 m**

Length of u/s pitching* = 2 [R.L. of u/s bed – R.L. of bottom of u/s cut–off]
= 2 [201.4 – 197.96] = 2 × 3.44 = 6.88 m; **say 6.9 m**

Similarly, length of d/s brick pitching
= 2 [R.L. of d/s bed – R.L. of bottom of d/s cut off]
= 2 [201.4 – 197.1] = 2 × 4.3 = 8.6 m.

The pitchings may be supported by toe walls, 0.4 m wide and 1 m deep, as shown in chart Fig. 14.36.

Example 14.5. *Suggest suitable cross-drainage works at the following crossings :*

(a) Irrigation channel

Discharge = 350 cumecs
Bed width = 28 m
FSD = 6.2 m
FSL = 204.3 m
CBL = 198.1 m
Natural ground level = 179.3

Natural Drainage

HFQ = 4300 cumecs
Drainage bed level = 194.9 m
HFL = 198.5 m

(b) Irrigation channel

Full supply discharge = 350 cumecs
Bed width = 24 m
FSD = 6.2 m
FSL = 215.6 m
CBL = 209.4 m
Canal bed slope = 1 in 10,000.

Natural Drainage

H.F.Q. = 390 cumecs
Drainage bed level = 207.4 m
HFL = 209.3 m
Drainage bed slope = 1 in 500
Springing level = 207.6 m

Solution. (*a*) In this case, the canal bed level (198.1 m) is higher than the drainage bed level (194.9) by 3.2 m and, therefore, we may think of taking the canal above the drainage. Further, drainage FSL (198.5 m) is above the canal bed (198.1 m), we will have to syphon the drainage water ; and thus a *syphon aqueduct* can be constructed at the site. However, since syphoning of drainage may not prove economical, and on the other hand, the canal bed may be raised slightly so as to pass drainage water freely below the canal trough (as the difference in CBL and drainage *HFL* is quite small *i.e.* 0.3 m). In that case, the minimum canal bed level to be kept at the crossing will be equal to

= Drainage HFL + thickness of canal trough slab + Min. free-board
= 198.5 + 0.6 + 0.9 = 200 m.

Therefore, bed of canal trough will have to be raised by an amount equal to 200 – 198.1 = 1.9 m at the site of the crossing. Hence, hump is provided along the length of trough with flat u/s slope and d/s slope to join to the natural canal bed levels on both sides. The crossing will then be designed as an **Aqueduct.** The design procedure as used in example 14.1 can be adopted.

*Although the canal is evidently a pitched canal as the velocity $\left(i.e. \frac{354}{30.2 \times 6.2} = 1.89 \text{ m/sec.}\right)$ is high, still these pitching lengths which would be necessary to be properly maintained for the safety of the structure have been worked out. Moreover, due to pitched canal, the scouring tendencies etc. will be less, however, for safety and conservative design purposes, the various data have been worked out assuming that in a worst case, the pitching of canal may get broken.

(*b*) In this case, the canal bed level (209.4 m) is higher than the drainage bed level (207.4 m) by about 2 m ; so we may think of taking the canal above the drainage. Further, the drainage HFL (209.3 m) is below the canal bed level (209.4 m) by only 0.1 m ; which is less than the free-board and slab thickness, hence, the drainage will have to be syphoned below the canal. The other possibility of raising the canal bed is not being adopted here, as the drainage is not too large in comparison with canal discharge.

The work-will, therefore, be designed as a **syphon aqueduct**, and design procedure adopted as in example 14.2.

14.6. Provision of Joints and Water Bars in R.C.C. Ducts of Aqueducts and Super Passages

R.C.C. box troughs (or ducts) carry water in aqueducts and super-passages, as explained in the previous articles. The box trough will span across the drain or the canal, rested at suitable intervals (spans) on piers or abutments. The construction of such R.C.C. troughs will involve various types of joints, such as *construction joints, expansion joints,* etc. Water will leak through these joints, unless proper arrangements are made to prevent such leakages. *Water bars* or *water-stops* are provided to stop such leakages through the joints. The water bars may be made of metals like copper sheets or galvanised iron sheets, or of rubbers (natural as well as synthetic), or of P.V.C., etc.

14.6.1. Types of Joints in R.C.C. Constructions. The various types of joints that may be involved in R.C.C. constructions can be divided into the following types :

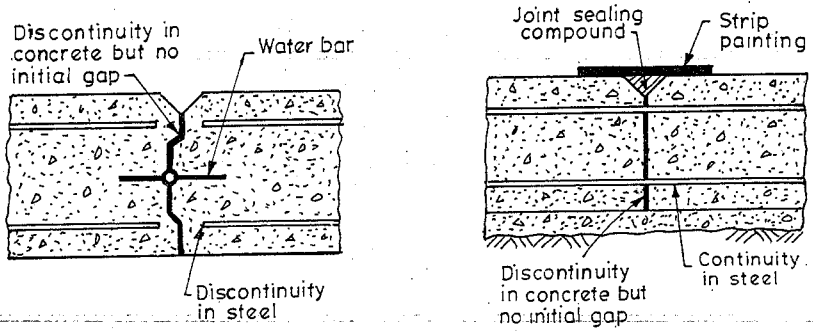
(1) *Movement joints* including the :

(*i*) *Contraction joints*, (*ii*) *Expansion joints*, and (*iii*) *Sliding joints* ; and

(2) *Construction joints.*

These joints are discussed below :

1 (*i*). **Contraction joints.** It is a movement joint with a deliberate discontinuity but no initial gap between the concrete on either side of the joint, the joint being intended to accommodate contraction of concrete. (See Fig. 14.37).



(*a*) Complete contraction joint (*b*) Partial contraction joint

Fig. 14.37. Typical contraction joints.

A distinction should also be made between a *complete contraction joint* (See Fig 14.37 *a*) in which both concrete and reinforcing steel are interrupted ; and a *partial contraction* joint (See Fig. 14.37 *b*) in which only the concrete is interrupted, the reinforcing steel running through.

1(*ii*). **Expansion joints.** A movement joint with complete discontinuity in both reinforcement and concrete, and intended to accommodate either expansion or contraction of the structure (See Fig. 14.38) is known as an expansion joint.

In general, such a joint requires the provision of an initial gap between the adjoining parts of a structure, which by closing or opening, accommodates the expansion or contraction of the structure. Design of the joint as to incorporate sliding surfaces is not, however, precluded and may sometimes be advantageous.

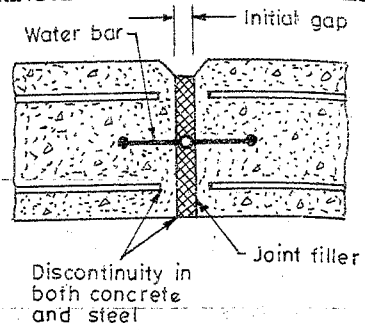


Fig. 14.38. A typical Expansion joint.

1(*iii*). **Sliding joints.** A movement joint with complete discontinuity in both reinforcement and concrete at which special provision is made to facilitate relative movement in place of the joint is known as a sliding joint. A typical application is in between a wall and a floor in same cylindrical tank designs (See Fig. 14.39)

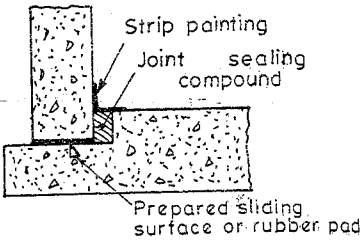


Fig. 14.39. A typical sliding joint.

(2) **Construction joints.** A joint in the concrete introduced for convenience in construction at which special measures are taken to achieve subsequent continuity without provision for further relative movement, is called a construction joint. A typical application is between successive lifts in pouring concrete in walls of a reservoir or a box trough. (See Fig. 14.40).

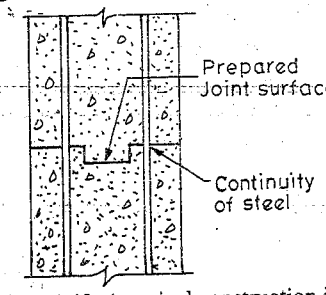


Fig. 14.40. A typical construction joint.
(Provision of water bar may also be made not shown here)

The position and arrangement of all the construction joints should be predetermined by the design engineer. *Consideration should be given to limiting the number of such joints, and to keeping them free from possibility of percolations in a similar manner for as for contraction joints.* Water bars may, therefore, be provided in the construction joints also, as is done in contraction joints (Fig. 14.37).

14.6.2. Spacing of Joints in R.C.C. Structures. Unless alternative effective means are taken to avoid cracks by allowing for the additional stresses that may be induced by temperature or shrinkage changes or by unequal settlement, movement joints should be provided at the following spacings :

(*i*) In reinforced concrete floors, movement joints should be spaced at not more than 7.5 m apart in two directions at right angles. The wall and floor joint should be in line, except where sliding joints occur at the base of the wall, in which case correspondence is not so important.

(*ii*) For floors with only nominal percentage of reinforcement (smaller than the minimum specified), the concrete floor should be cast in panels with sides not more than 4.5 m.

(*iii*) In concrete walls, the vertical movement joints should normally be placed at a maximum spacing of 7.5 m in reinforced walls and 6 m in unreinforced walls. The maximum length desirable between vertical movement joints will depend upon the tensile strength of the walls, and may be increased by suitable reinforcement. Thus when a sliding layer is placed at the foundation of a wall, the length of wall that can be kept free of cracks depends upon the capacity of wall section to resist the friction induced at the plane of sliding. Approximately, the wall has to stand the effect of a force at the plane of sliding equal to weight of half the length of wall multiplied by the coefficient of friction.

(*iv*) Amongst the movement joints in floors and walls as mentioned above, expansion joints should normally be provided at a spacing of not more than 30 m between successive expansion joints or between the end of the structure and the next expansion joint, all other joints being of the contraction type.

(*v*) When, however, the temperature changes to be accommodated are abnormal or occur more frequently than usual as in the case of storage of warm liquids or in uninsulated roof slabs, a smaller spacing than 30 m should be adopted, (that is a greater proportion of the movement joints should be of the expansion type). When the range of temperature is small, for example, in certain covered structures, or where restraint is small, for example, in certain elevated structures, none of the movement joints provided in small structures upto 45 m length need be of the expansion type. Where sliding joints are provided between the walls and either the floor or roof, the provision of movement joints in each element can be considered independently.

14.6.3. Width of Gap in Expansion Joint. An expansion joint requires the provision of an initial gap between the concrete faces on the two sides of the joint. The initial width of this gap should be sufficient to accommodate freely the maximum expansion of the structure.

In determining this initial width, consideration should be given to the requirements of the jointing materials. These will normally require the maintenance of certain minimum width of gap during maximum expansion of the structure. The joint should also be suitably treated as to maintain water-tightness during movement of the joint.

14.6.4. Jointing Materials. Jointing materials may be classified as follows :

(*i*) *joint fillers* ;

(*ii*) *Water bars* and *joint cover plates*; and

(*iii*) *Joint sealing compounds* (including primers where required)

They are discussed below :

(*i*) **Joint fillers.** Joint fillers are usually compressible sheet or strip materials used as spacers. They are fixed to the face of the first placed concrete and against which the second placed concrete is cast. With an initial gap of about 30 mm, the maximum expansion or contraction that the filler materials may allow may be of the order of 10 mm.

Joint fillers, as at present available, can not by themselves function as watertight expansion joints. They may be used as support for an effective joint sealing compound

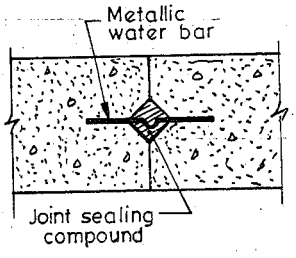
in floor and floor joints. But they can only be relied upon as spacers to provide the gap in an expansion joint, the gap being bridged by a water bar (See Fig. 14.42).

(*ii*) **Water bars.** Water bars or *water stops* are the preformed strips of impermeable materials which are embedded in concrete during construction, so as to span across the joint, as to provide a permanent water tight seal during the whole range of joint movement.

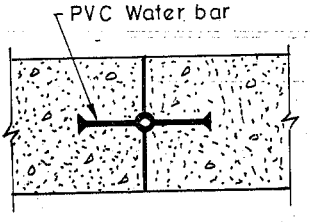
The water bars are usually made of *metal* sheets like copper, galvanised iron etc. ; or may be made of *rubber* (natural or synthetic) ; or of plastics like *P.V.C.* (Polyvinyl chloride). *Metal bars* can be used in water conveyance structures or dams, where the foundations are not expected to yield appreciably ; otherwise they will snap. *Annealed copper* is the most common metal used for water stops, in thickness not less than 1.5 mm; but it is likely to become brittle in course of time and may then crack. *Monel metal* is considered more durable though sometimes 20 gauge *galvanised iron* or *steel* strips are also used.

The most usual shape of metal bars is in the form of strips with central longitudinal corrugation, to obtain a V, U or M shape. The total width of the water bar may range from 150 mm to 300 mm. For box aqueducts etc., 180 to 225 mm wide water bar may suffice; while for dams, 300 mm wide bars may be preferred. *Rubber water bars* are, however, preferred to metal bars for their economy, and suitability to some what yielding foundations. Metal and rubber water bars are invariably used in ducts of aqueducts and super-passages in expansion joints; *PVC water bars*, on the other hand, are, usually adopted in contraction and construction joints.

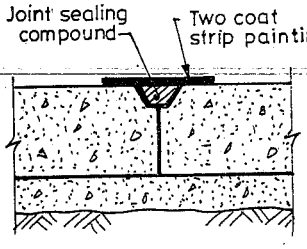
Typical use of various types of water bars in contraction or construction joints are shown in Fig. 14.41 (*a*) to (*d*); whereas Fig. 14.42 (*a*) to (*g*) shows the use of various type of water bars in expansion joints.



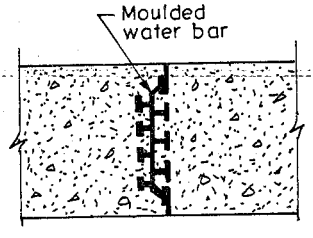
(*a*) metal water bar with central corrugation



(*b*) PVC water bar



(*c*) metal water bar with two coat strip painting



(*d*) moulded water bar

Fig. 14.41. Typical details showing use of water bars surrounded with joint sealing material in Contraction Joints.

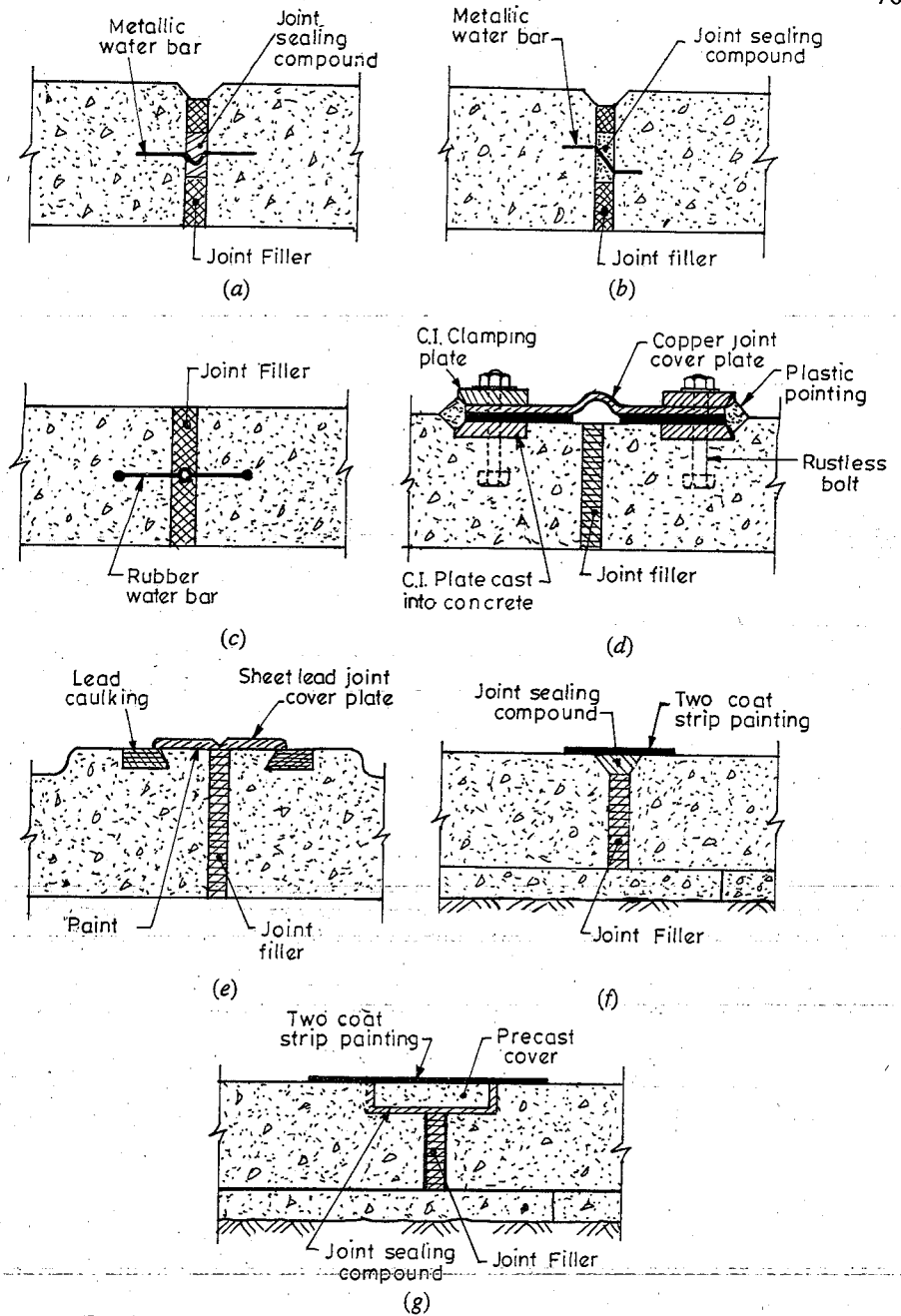
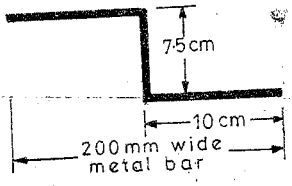
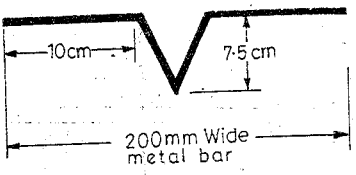


Fig. 14.42. Typical details showing use of various types of water bars and jointing materials in Expansion joints.

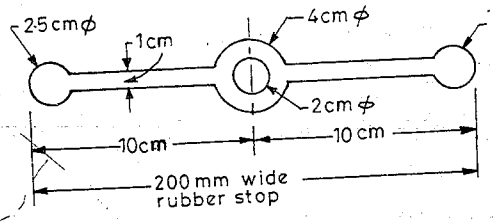
Most Commonly used dimensions & shapes of *metal water stops* (Z-shape and V-shape), *rubber water stop* (dumb well with central bulb type shape) for expansion joints; and of PVC water stop for construction joints of aqueducts and super-passages, are shown in Fig 14.43 (*a*) to (*d*).



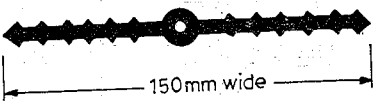
(*a*) 200 mm wide Z-shape metal water stop for expansion joints



(*b*) 200 mm wide V-shape metal water stop for expansion joints.



(*c*) 200 mm wide dumb well with central bulb type rubber water stop for expansion joints



(*d*) 150 mm wide *PVC* water stop serrated with central bulb for construction joints.

Fig. 14.43. Dimensions and shapes of commonly used water bars for ducts of Aqueducts and Super-passages.

With all water bars, it is important to ensure proper compaction of the concrete. The bar should have such shape and width that the water path through the concrete round the bar should not be unduly short.

The holes sometimes provided on the wings of copper water bars to increase bond, shorten the water path and may be disadvantageous. *The water bar should either be placed centrally in the thickness of the wall ; or its distance from either face of the wall should not be less than half the width of the bar. The full concrete cover to all reinforcement should be maintained.*

The strip water bars at present available in the newer materials need to be passed through the end shutter of the first-placed concrete. It can be appreciated, however, that the use of the newer materials make possible a variety of shapes or sections. Some of these designs, for example, those with several projections (*see* Fig. 14.41*d*), would not need to be passed through the end shutter and by occupying a bigger proportion of the thickness of the joint, would also lengthen the shortest alternative water path through the concrete.

(*ii b*) **Joint Cover Plates.** Joint cover plates are sometimes used in expansion joints to avoid the risk of a fault in an embedded water bar. The cover plate may be of copper or sheet lead. If copper cover plate is used, it should be clamped to the concrete face on each side of the joint using suitable gaskets to ensure water tightness (*see* Fig. 14.42*d*). If sheet lead is used, the edges may return into grooves formed in the concrete and be made completely watertight by lead caulking (*see* Fig. 14.42 *e*). Faces of the concrete to which sheet lead is to be fixed should be painted with bituminous or other suitable composition, and the lead sheet should be similarly coated before fixing.

(*iii*) **Joint Sealing Compounds.** Joint sealing compounds are impermeable ductile materials which are required to provide a watertight seal by adhesion to the concrete

throughout the range of joint movement. The commonly used materials are based on asphalt, bitumen, or coal tar pitch with or without fillers, such as limestone or slate dust, asbestos fibre, chopped hemp, rubber or other suitable material. These are usually applied after construction or just before the reservoir is put into service by pouring in the hot or cold state, by trowelling or gunning or as performed strips ironed into position. They may also be applied during construction, such as by packing round the corrugation of a water bar. A primer is often used to assist adhesion and some local drying of the concrete surface with the help of a blow lamp is advisable. The length of the shortest water path through the concrete should be extended by suitably painting the surface of the concrete on either side of the joint.

The main difficulties experienced with this class of material are in obtaining permanent adhesion to the concrete during movement of the joint, whilst at the same time ensuring that the material does not slump or is not extruded from the joint.

In floor joints, the sealing compound is usually applied in a chase formed in the surface of the concrete along the line of the joint (*see* Fig. 14.42*c*). The actual minimum width will depend on the known characteristics of the material. In the case of an expansion joint, the lower part of the joint is occupied by a joint filler (*see* Fig. 14.42*f*). This type of joint is generally quite successful, since retention of the material is assisted by gravity and, in many cases, sealing can be delayed until just before the reservoir is put into service so that the amount of joint opening subsequently to be accommodated is quite small. The chase should not be too narrow or too deep to hinder complete filling and the length of the shortest water path through the concrete should be extended by suitably painting the surface of the concrete on either side of the joint. Here again a wider joint demands a smaller percentage distortion in the material.

An arrangement incorporating a cover slab, similar to that shown in Fig. 14.42 g, may be advantageous in reducing dependence on the adhesion of the sealing compound in direct tension.

Use of sealing compounds for vertical joints is not very successful. A stepped-joint instead of a straight through-joint with a water bar incorporated in the joint and sealing compound packed round the corrugation of the water bar would be much more successful.

Complete typical details of a 30 mm wide expansion joint adopted in the construction of an Aqueduct are shown in Fig. 14.44, where two water stops (225 mm wide rubber

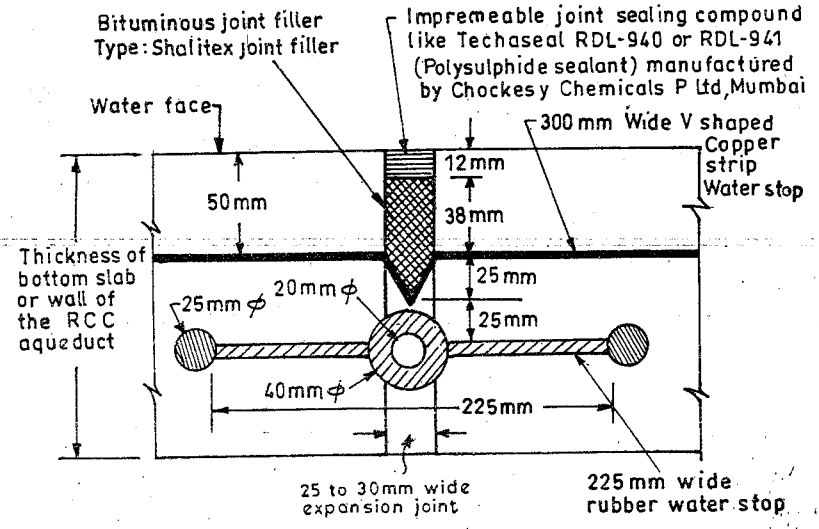


Fig. 14.44. Details of typical Expansion joint in the base slab and walls of R.C.C. duct or trough.

water stop and 300 mm wide V-shaped copper water stop) were used to ensure complete water-tightness.

PROBLEMS

1. (*a*) What is meant by a "Cross-Drainage Works" ? Explain as to why such works are not met within a ridge canal system. (*Madras University, 1973*)

(*b*) Write short notes on :

(*i*) Syphon

(*ii*) Super passage

(*iii*) Syphon aqueduct. (*Madras University, 1973*)

2. Write short notes on :

(*i*) Aqueduct

(*ii*) Syphon aqueduct

(*iii*) Canal syphon (*Madras University, 1972*)

(*iv*) Level crossing.

3. Design and give a dimensional sketch of an aqueduct to carry water of an earth canal over a drainage with the following data :

RL of bed of drainage	= 520.00 m
HFL of drainage	= 523.00 m
Bed width of drainage	= 50.00 m
Side slopes of drainage at crossing	$= \frac{1}{2} : 1$
R.L. of ground	= 525.00 m
R.L. of bed of canal	= 524.50 m
Discharge of canal	= 30 cumecs
Depth of water in canal	= 1.70 m
Bed width of canal	= 22.0 m.

4. (*a*) Name the different types of cross drainage works. Explain how you would avoid one type of cross drainage work and prefer to adopt another type by simply changing the alignment of the canal taking off from a head-work. (*Engg. Services, 1974*)

(*b*) Under what conditions of drainage and canal crossings are syphons provided ? Draw a plan and section through a typical branch canal syphon, and suggest a method for reducing uplift on the floor of the work.

5. (*a*) What are the different types of cross drainage works that are necessary on a canal alignment? State briefly the conditions under which each one is used. (*Madras University, 1974*)

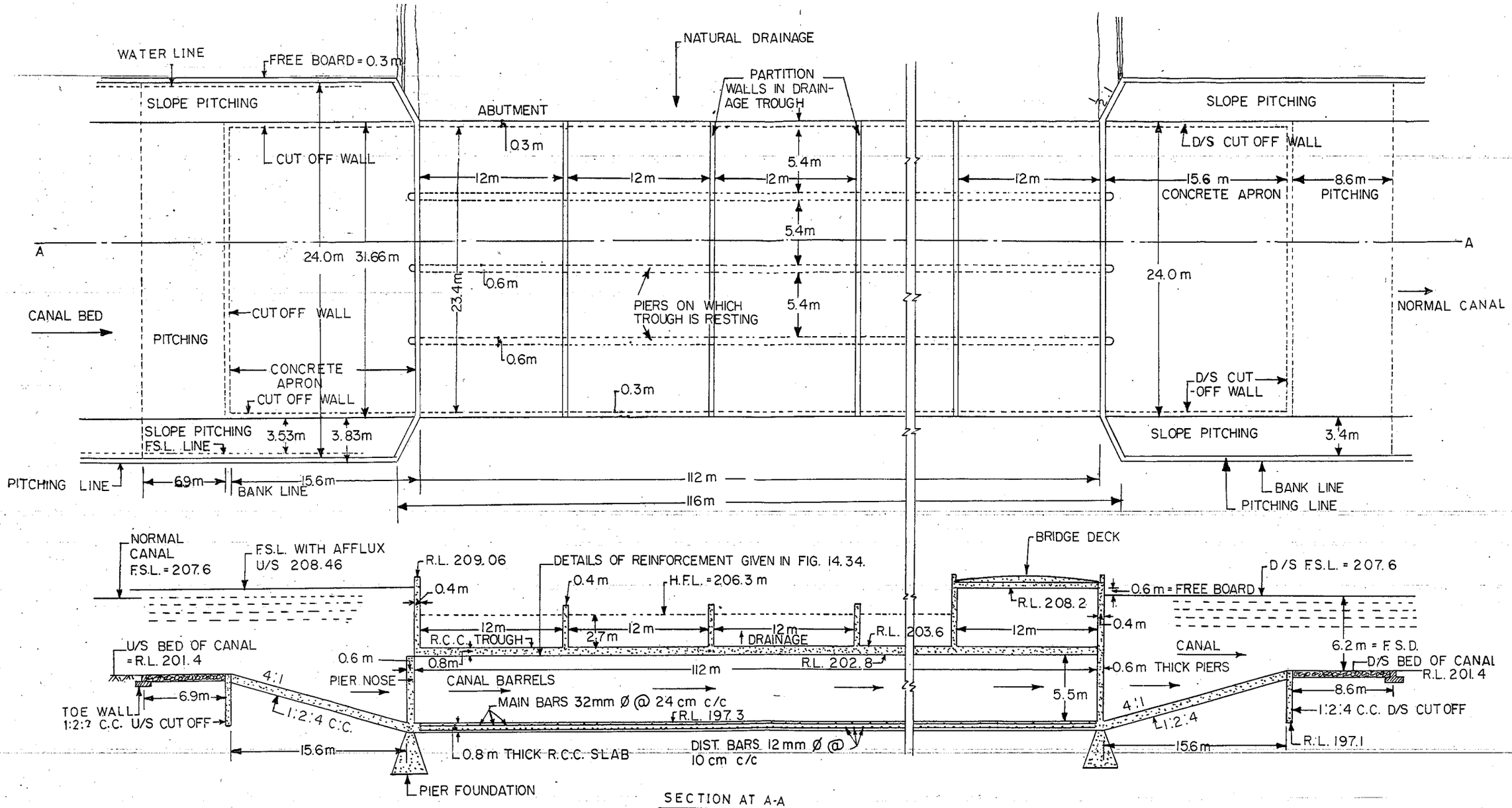
(*b*) Discuss with neat sketches, the three different types of aqueducts which can possibly be constructed depending upon the size of the drainage to be passed below the canal. Also discuss the factors governing the choice of any of these three types of aqueducts.

6. Show by rough sketches the type of cross drainage work suitable for each of the following cases:

(*a*)

	Drain	*Canal*
Discharge in cumecs	200	20
BL	25.00 m	30.00 m
FSL		31.50 m
HFL	28.00 m	

[**Ans.** Aqueduct of Fig. 14.3 is recommended]



SECTION AT A-A

Fig.14.36

(*b*)

	Drain	*Canal*
Discharge in cumecs	2 cumecs	400 cumecs
BL	52.2 m	48.00 m
FSL		53.00 m
HFL	53.2 m	

[**Ans.** Syphon of Fig. 14.7 is recommended]
(*Madras University, 1976*)

7. Discuss the various types of cross drainage works used in canal systems. What considerations govern the selection of the different types of works, mainly depending upon the levels of the canal and the drainage. Illustrate by drawing a neat sketch of each type of structure.

8. Give neat sketch of suitable designs of aqueducts for each of the following crossings :

(*a*) A major canal over a small drainage. [**Ans.** Aqueduct of type I, Fig. 14.10]

(*b*) A canal carrying low discharge over a large drainage. [**Ans.** Aqueduct of Type II, Fig. 14.11]

(*c*) A major canal over a large drainage. [**Ans.** Aqueduct of Type III, Fig. 14.12]

Explain in detail the method of design of fluming required in the crossing at (*c*) above.

9. (*a*) State under what circumstances you will recommend the use of the following cross drainage structures :

(*i*) Syphon

(*ii*) Inlet

(*iii*) Aqueduct

(*b*) Following data were collected from a syphon aqueduct :

Diameter of the barrel (single)	= 3 m
Length of the barrel	= 90 m
Discharge through the barrel	= 25 cumecs
Friction factor (in Darcy-Weisbach formula)	= 0.013
Coefficient of bend loss (2 bends)	= 0.10
Coefficient of head loss in expansion at outlet	= 0.20
Coefficient of head loss in contraction at inlet	= 0.10.

Determine afflux. Neglect velocity head in drainage channel. (*AMIE, 1970*)

[Solution :

$$\text{Total head loss} = h = \underbrace{0.10\left(\frac{V^2}{2g}\right)}_{\text{entry loss + bend loss}} + \underbrace{2\times 0.10\left(\frac{V^2}{2g}\right)}_{\text{outlet expansion loss}} + \underbrace{0.2\left(\frac{V^2}{2g}\right)}_{} + \underbrace{\frac{f' LV^2}{2gd}}_{\text{friction loss in pipe}}$$

$$= 0.5\frac{V^2}{2g} + \frac{0.013\times 90}{3}\frac{V^2}{2g} = 0.89\frac{V^2}{2g}$$

$$\text{Velocity in the barrel} = \frac{Q}{A} = \frac{25}{\frac{\pi}{4}\times(3)^2}\ \text{m/sec.} = 3.52\ \text{m/sec.}$$

$$\therefore \qquad h = 0.89\times\frac{(3.52)^2}{2\times 9.81} = 0.57\ \text{m.}$$ **Ans.]**

10. (*a*) What are cross-drainage works ? What is the necessity of such a work in a canal project, and how does this necessity is fulfilled by such works ?

(*b*) Enumerate the different methods which may be used for designing the canal transitions for a flumed canal, and the conditions under which each can be used. Describe in details a method of designing canal transitions when water depth may or may not remain constant.

11. (*a*) What is meant by cross drainage works and what is their importance in a canal project ?

(*b*) Describe briefly the step by step procedure that you will adopt for designing an unflumed syphon aqueduct.

12. Design a syphon aqueduct with the following data :

For Canal

Discharge	= 56 cumecs
Bed width	= 32 m
F.S. depth	= 1.98 m
R.L. of bed	= 267.00 m

For Drainage

High flood discharge	= 425 cumecs
HFL	= 268.20 m
General bed level of low water cross-section	= 265.50 m
General ground level	= 267.20 m

13. Design a suitable cross drainage work, if the crossing in problem 12 above is shifted upstream along the drainage so that the HFL is 269.55 m, the bed level of the drainage is 267.00 m, and general ground level is 269.05 m. The canal data remaining the same.

15

*Construction of Culverts and Small Road Bridges Across Drains and Canals**

15.1. Introduction

Whenever a new road has to cross an existing drain or a canal, or sometimes, when a drain has to cross an existing road ; a small bridge or a culvert** is constructed at the point of crossing. The number of such culverts required in any road or canal project is generally very large and account for a large percentage of the total expenditure on the project. These small bridges or culverts should therefore, be designed safely and economically, so as to avoid unnecessary over investments. In the pages below, we shall, therefore, discuss the important considerations that are involved in their designs.

15.2. Data Collection

After a proper site of construction has been decided on some straight reach of the stream, the highest flood level at the site should be determined. The longitudinal section of the stream extending by about 200 to 1500 metres upstream and downstream should be plotted depending on the catchment area of the stream (at the point of construction) varying from 250 to 1240 hectares or more. The actual flow velocity may be observed during times of floods, which may provide a good check on the velocity calculated theoretically.

15.3. High Flood Discharge Computations

After collecting the required necessary data, the maximum discharge that is likely to reach at the site may be calculated by using any of the various methods given in article 7.9. The various empirical formulas given for this purpose should only be used by experts and a new designer should try to use the Rational formula only.

Road bridges and culverts are normally designed for 1 in 50 year frequency discharge, although important rail bridges may be designed for 1 in 100 years discharge.

After the design flood discharge has been finalised, the total span and number of spans of the bridge should be decided, as given in articles 15.4 and 15.8.

15.4. Linear Waterway of the Bridge

The linear waterway of a bridge across a purely alluvial stream is generally kept equal to its regime width (W) as given by Lacey. The Lacey's regime width of the stream is taken equal to its wetted perimeter (P), given by the equation,

$$W = P = 4.75\, Q^{1/2}$$

$$\therefore \quad L = 4.75\, Q^{1/2} \qquad \ldots(15.1)$$

where Q is the dominant discharge of the river

* The discussions here are based on Indian Standards laid down by Indian Road Congress in its various publications which are hereby duely acknowledged.

** A bridge with span upto 8 m is called a culvert.

Hence, the existing width of the stream is contracted at the bridge site by upstream wing walls (or training works) so as to make it equal to W, and the bridge may then be spanned across it. The contraction of the stream up to the regime width leads to economy, as the bridge length is reduced ; but further contraction beyond regime width are generally not economical, because any contraction beyond W (Lacey's regime width) will increase the discharge intensity through the bridge spans, and thus increasing the Lacey's regime scour depth (R). Due to the increase in scour depth, the foundations of the bridge piers, abutments, and cut off walls will have to be taken deeper. The necessity of providing deeper foundations in streams contracted beyond Lacey's regime width, will increase the cost of foundations, although the length of the bridge superstructure will reduce. *Much of the saving in cost expected from decreasing the length of the bridge in such a case will, therefore, generally be eaten up by the corresponding increase in the depth of the foundations and the size of the training works.* Hence, in alluvial streams, it is generally futile to contract the waterway beyond the regime width, as no appreciable saving can be obtained.

However, in streams which are not alluvial and are either semi-alluvial (sides rigid and bed alluvial) or purely rigid, the waterway may be chosen equal to the actual surface width of the stream measured from edge to edge of water along the H.F.L., on the plotted cross-section. The waterway in purely rigid streams may be kept still less, if appreciable saving is obtained and afflux is not detrimental. For streams which overflow their banks and create very wide surface widths with shallow side sections, the bridge may span the active channel, but this waterway should not be so low as to cause excessive detrimental afflux. The waterway, in such a case has, therefore, to be chosen by judgement and intelligence of the designer with these guidelines.

Note. When the natural available width of the stream is found nowhere near the value given by Lacey's equation. (*i.e.* $W = 4.75\sqrt{Q}$), we must expect to find that the banks, even though not rocky, are not friable enough to be treated as incoherent alluvium and hence, the stream is no longer a stream in alluvium. Such streams may then be *treated as semi-alluvial streams.*

15.5. Scour Depth Computations

It was explained earlier in chapter 4, that, if a constant discharge passes through a straight stable reach of an alluvial stream for an infinite time, the boundary of its cross-section would ultimately become elliptical. This will happen when the stream has achieved a state of regime, and in such a case, the depth in the middle of the stream would be equal to the normal scour depth. This **normal depth of scour** (*i.e.* **Lacey's regime scour depth**) is generally taken as a guideline for fixing the depth of the foundations.

The *Lacey's regime scour depth* (R_r') is calculated as given below :

(*a*) For a purely alluvial stream having a span length (L) equal to W (given by $W = 4.75\sqrt{Q}$).

$$\text{Regime scour depth} = R_r' = 0.473\left(\frac{Q}{f}\right)^{1/3} \qquad ...(15.2)$$

where f is obtained by the equation

$$f = 1.76\sqrt{d_{mm}},\ d_{mm} \text{ being average particle size in mm} \qquad ...(15.3)$$

(*b*) For *purely alluvial streams* which are contracted beyond W at the bridge site, the value of Lacey's Scour depth (R') may be computed by using the formula :

$$\text{Scour depth} = R' = 1.35\left(\frac{q^2}{f}\right)^{1/3} \qquad ...(15.4)$$

where q is the discharge intensity per unit width of the stream at bridge site, and equals Q/L, where L is the width of the stream at the bridge site.

R' can also be worked out by using the formula

$$R' = \left(\frac{W}{L}\right)^{0.61} \times R_r' \qquad ...(15.5)$$

where W = the Lacey's regime width for alluvial streams

R_r' = the Lacey's regime scour depth given by Eq. (15.2)

Note. The above calculated values of R_r' and R' are depths below water surface elevation of the stream.

(*c*) For *quasi-alluvial streams* (*i.e.* bed is not pitched and sides are banked with stable soils or pitchings), having the bridge span length equal to the length of water line between banks, measured along HFL (called natural unobstructed width of the stream W'), the normal secoured depth (D') may be computed by using the formula :

$$D' = \frac{1.21\,Q^{0.63}}{f^{0.33}\,W'^{0.60}} \qquad ...(15.6)$$

where D' = the scour depth in quasi alluvial streams having $L = W'$

W' = the natural unobstructed width of the stream, as shown in Fig. 15.1.

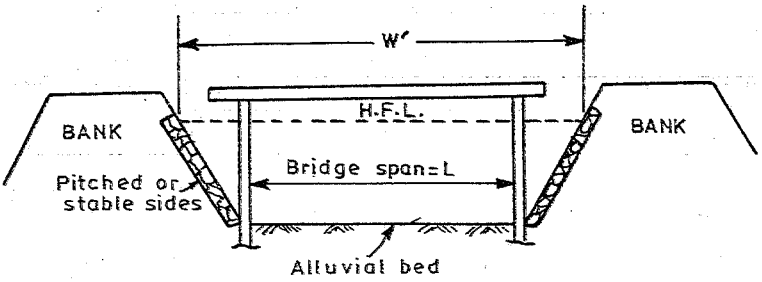


Fig. 15.1. Semi-Alluvial streams.

(*d*) For *quasi-alluvial streams which are contracted* so as to keep L lesser than W', the normal scour depth (D) is given by :

$$D = D'\left(\frac{W'}{L}\right)^{0.61} \qquad ...(15.7)$$

where D' is given by Eq. (15.6).

(*e*) For *purely rigid streams* (such as lined pucca drains), however, from theoretical considerations, there will be no scouring, and the foundations of the bridge sub-structure

will have to be taken below the bed, not by scour depth considerations, but only by nominal amount, so as to provide sufficient grip on them.

15.6. Maximum Scour Depth Computations

In natural streams, the scouring is not uniform along the bed width. It is not uniform even in straight reaches. Particularly at the bends and also around obstructions to the flow (*e.g.*, the piers of a bridge), there is a deeper scour than the normal scour. The maximum possible scour depth should, therefore, be fairly estimated and the foundations of the structure marginally taken below that level, so as to obtain a factor of safety on that maximum scour. The maximum scour depth is generally taken as x times the normal scour depth R. The value of x is generally taken as 1.5 for a single span structure (with no piers) on a straight reach of the stream, and is taken as 2.0 on bad sites on curves or for multispan structures.

15.7. Depth of Bridge Foundations (D_f)

The foundations of bridges on erodibe beds are taken down below the H.F.L. by an amount equal to 4/3 times the maximum scour depth, or equal to maximum scour depth plus 2 metres for arched bridges and 1.2 metres for other bridges, whichever is more.

This formula is applicable only when no bed floor is provided under the structure and the stream is free to scour as it may. The structure is then said to have deep foundations and it is always considered better to have such foundations as far as possible. But in certain cases, especially on small culverts, bed floors are generally provided. In such a case, the following in considered a safe practice on erodible beds :

"Keep the top of the floor about 0.3 metre below the bed level. Take the foundations of the abutments 1.2 metres below the top of the floor. Provide an upstream curtain wall 1 to 1.5 metres deep and down-stream curtain wall 1.5 to 2.5 metres deep from the top floor depending on the velocity of flow through the structure and erodibility of the bed materials."

These rules are based on the I.R.C. Code of Practice for Road Bridges.

15.8. Total Clear Span and Number of Spans

As a fundamental rule, the number of spans should be as less as possible, since piers obstruct flow. It is particularly important in hilly regions where torrential velocities prevail, and is always better to span from bank to bank using no peirs if possible.

By increasing the length of one span, the number of spans can be reduced. In such a case, the cost of super-structure increases and that of sub-structure decreases. *The most economical span length is one for which the cost of sub-structure is equal to the cost of super-structure.* This economy should, therefore, be worked out for large bridges and where difficult conditions are anticipated in sinking of foundations. Any deviations from the ideal economic case has to be justified strongly. When the regulators, barrages, etc. are provided with bridges, the span is generally guided by the economy as well as the availability of gate sizes.

Length of one span. The economical length of one span say (l) may be chosen as per the following guidelines :

(a) For R.C.C. slab bridges

$$l = 1.5\,H \qquad ...(15.8)$$

where H = Total height of abutment or pier from the bottom of its foundations to its top
= D_f + Vertical clearance*, *i.e.* free-board
l = Clear span length.

(*b*) *For masonry arch bridges*

$$l = 2H \qquad ...(15.9)$$

where H = Total height between the bottom of the foundations to the intrados of the key stone
l = Clear span length.

Number of spans. When L is more than the economical span length (l), the number of spans required (N) is tentatively found by using

$$L = N.l \qquad ...(15.10)$$

Since N must be a whole number (preferably an odd number), l may have to be modified suitably. Varying span lengths may sometimes be adopted, so as to keep as close as possible to the requirements of economy and to cause least obstruction to the flow.

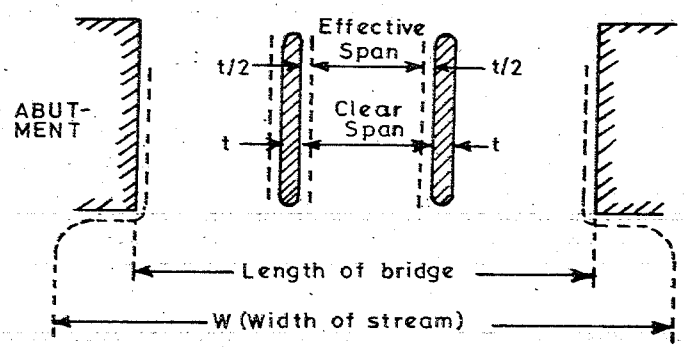


Fig. 15.2. (*a*) Plan at a Bridge site.

Effective clear span. It is assumed that each bridge pier causes contraction in the filaments of water equal to half its thickness on each side, and the obstruction at the ends due to abutments or pitched slope is ignored. Hence, the finally selected number and lengths of spans should be such that the sum of all the effective span lengths should be equal to the required linear waterway (L), and this will be equal to

$$L = \Sigma \text{ [of all the effective spans]}$$
$$= \Sigma \text{ [of all the clear span} - \text{Thickness of piers]}$$
$$L = \Sigma \text{ of all the clear spans} - \Sigma \text{ thickness of piers} \qquad ...(15.11)$$

The pier thickness above is taken as the mean value for the height of the pier from HFL to maximum scour depth.

* See Fig. 15.2 (*b*).

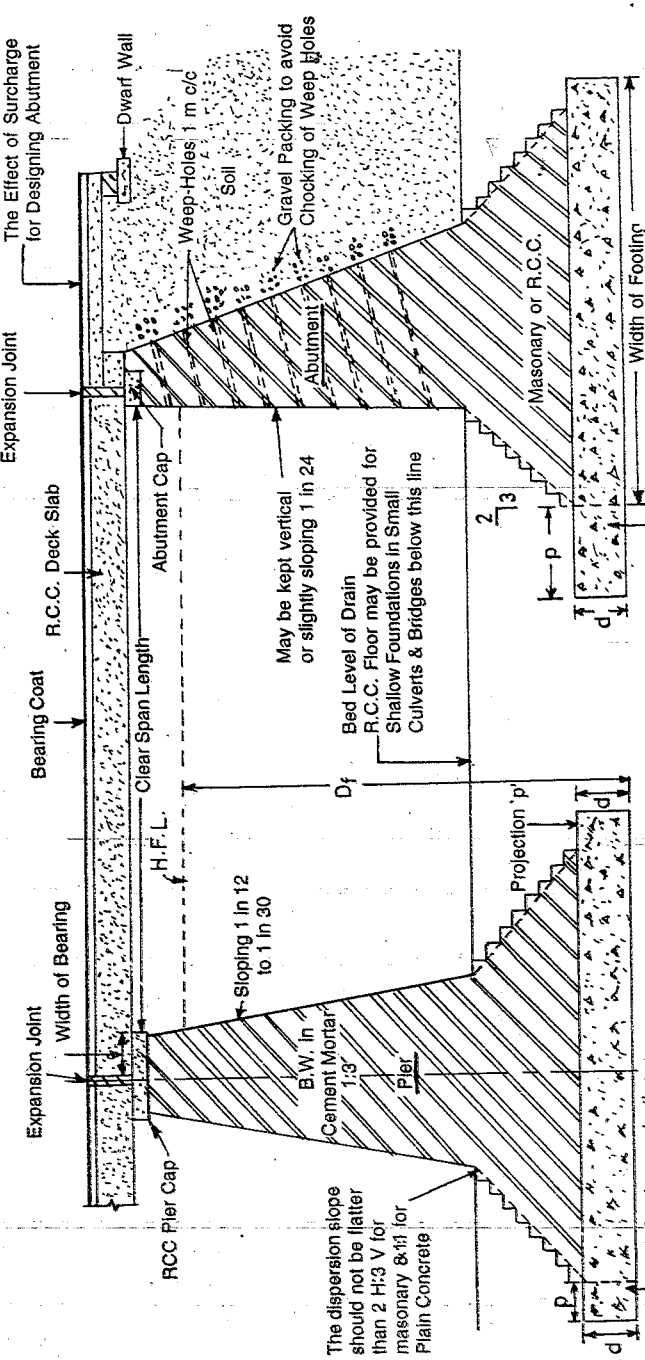


Fig. 15.2. (*b*) Section of abutment and piers for a slab bridge.

15.9. Vertical Clearance and Some Other Tentative Dimensions

They may be fixed as per I.R.C. codes for road bridges as follows :

Table 15.1. Value of Vertical Clearance

Discharge in cumecs	*Vertical clearance or Free-board in metres*
Below 0.3	0.15
0.3 to 3	0.3
3 to 30	0.6
30 to 300	0.9
300 to 3000	1.2
Above 3000	1.5

Table 15.2. Minimum Top widths of PCC or RCC Piers and Abutments

Clear span in metres	*Width of abutment in metres*	*Width of pier in metres*
3	0.38	0.45
6	0.45	0.55
9	0.55	0.65
12	0.65	0.75
15	0.75	0.88

Table 15.3. Minimum Top Widths of Masonry Piers and Abutments

Span	*3 m*	*6 m*	*12 m*	*24 m*	*40 m*	*50 m and above*
Top width of piers carrying simply supported spans (in m)	0.5	1.0	1.2	1.6	2.0	2.2
Top width of abutment and of piers carrying continuous spans (in m)	0.4	0.75	1.0	1.3	1.7	1.9

15.10. Afflux Computations

When a bridge is constructed across a contracted stream, water on the upstream will rise up. The maximum rise in water level near the bridge site is nothing but afflux. The greater is this afflux, lesser will be the available clearance for the same deck level. Since the 'clearance' is the distance between the u/s H.F.L. and the bottom of the bridge slab, it is nothing but freeboard. Hence, it is necessary to calculate afflux to examine its effects on clearance and also upon the regime of the channel u/s of the bridge, and also to determine the top levels of training walls, etc.

The afflux (h), the discharge (Q), the unobstructed stream width (W), and the provided linear waterway of the bridge (L) are all interconnected. Greater is the reduction in the linear waterway, the greater is the afflux. Since the downstream depth is not affected by the bridge, as the same is governed by the hydraulic characteristics of the d/s channel, it can be safely assumed that the u/s depth which prevailed before the bridge construction is the same as the d/s depth (y_d) that prevails even after the bridge construction.

Hence, y_d is the depth that prevailed at the bridge site before the construction of the bridge. To estimate afflux, we must know y_d. This can be found from the available gauge discharge curve of the stream, or may be calculated by the hydraulic parameters of the channel.

The discharge passing through the bridge openings can be calculated in the following different ways :

(1) **By Broad Crested Weir formula.** This formula is applicable so long as the afflux, *i.e.* [upstream depth (y_u) — downstream depth (y_d)] is not less than $\frac{1}{4} \cdot y_d$. In this formula, the discharge Q is dependent only upon y_u and is independent of y_d. The fact that the downstream depth y_d is having no effect on the discharge, nor on the upstream depth y_u, when the afflux is not less than $\frac{1}{4}y_d$, is due to the formation of the 'standing wave'.

The discharge Q is then given by

$$Q = 1.71 \cdot C_d L \left[y_u + \frac{V_a^2}{2g} \right]^{3/2} \qquad ...(15.12)$$

where $\frac{V_a^2}{2g}$ = head due to velocity of approach.

y_u = upstream water depth

C_d is the coefficient of discharge and accounts for frictional losses. The various values of C_d for different types of openings are given in Table 15.4.

Table 15.4. Values of C_d for Different Types of Bridge Openings

Type of bridge Opening	*Value of C_d*
Narrow bridge opening with or without floors	0.94
Wide bridge opening with floors	0.96
Wide bridge opening with no bed floors	0.98

(2) **By the Orifice Formula.** When the downstream depth is more than 80% of the upstream depth (*i.e.* the afflux is less than $\frac{1}{4}y_d$), the weir formula is not valid, as the performance of the bridge opening gets affected by the downstream depth (y_d). In such a case, the discharge can be calculated by using the Orifice formula given by

$$Q = C_0 \sqrt{2g} \cdot L \cdot y_d \cdot \left[h + (1+e) \frac{V_a^2}{2g} \right]^{1/2} \quad ...(15.13)$$

The values of coefficients e and C_0 may be taken from the curves given in Figs. 15.3 and 15.4.

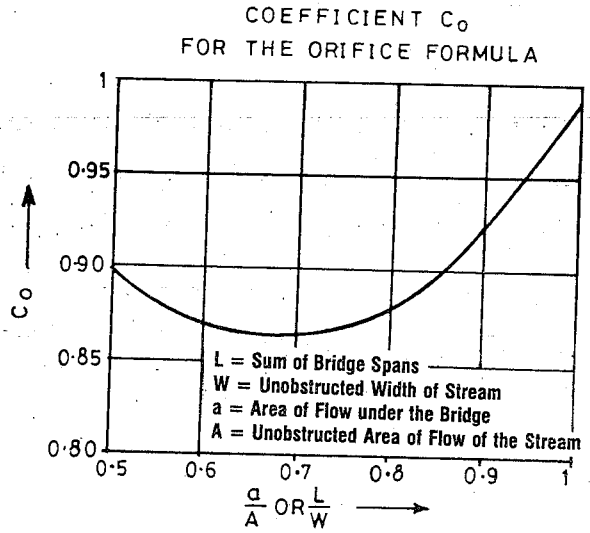


Fig 15.3

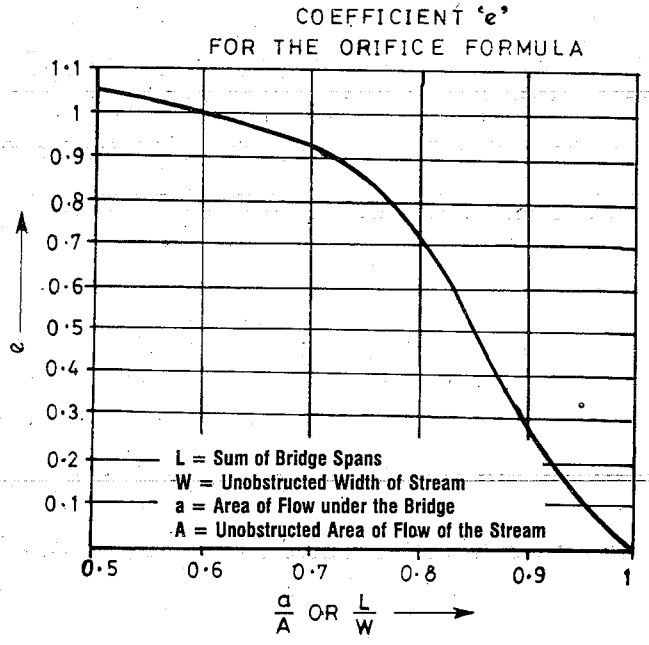


Fig 15.4

The derivations of these two formulas are given below :

Derivation of broad crested weir formula applied to bridge openings

With reference to Fig. 15.5, we have

The total energy (H) at section 1-1.

$$= y_u + \frac{V_a^2}{2g}$$

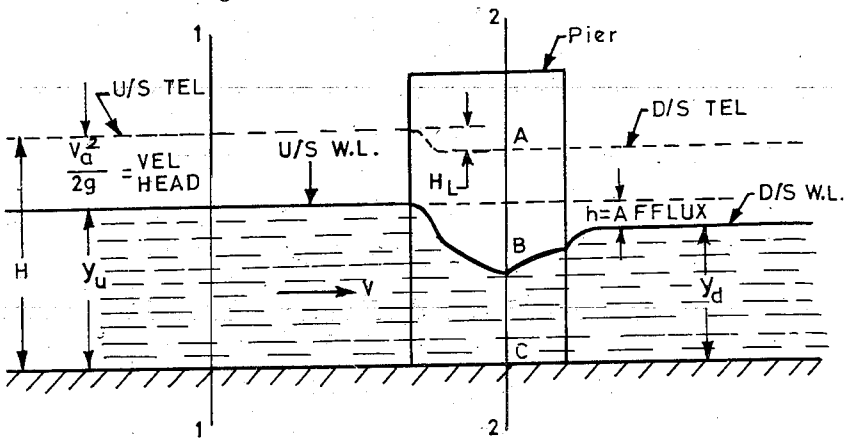


Fig. 15.5.

At section 2-2, let the velocity head AB is a certain fraction of H, say let it be $m \cdot H$.

$$\frac{V^2}{2g} = mH$$

where V is the velocity through the bridge opening.

$$\therefore \quad V = \sqrt{2g \cdot m \cdot H} \qquad ...(i)$$

Now, if the head loss due to entry and friction (*i.e.* H_L) is ignored, then by equating total energies at 1-1 and 2-2, we get

$$H = AB + BC = m \cdot H + BC$$

$$\therefore \quad BC = H - m \cdot H = H(1 - m)$$

The area of flow at section 2-2

$$= BC \times \text{Linear waterway} = BC \times L = H(1 - m) \cdot L$$

$\therefore$ Discharge through the bridge

$$(Q) = \text{Area} \times \text{Velocity} = H(1 - m)\, L \times V \qquad ...(ii)$$

But from (i) we have

$$V = \sqrt{2g \cdot mH}$$

$$\therefore \text{ Discharge } (Q) = H(1 - m) \quad L \cdot \sqrt{2gmH}$$

To account for losses at entry and friction, we introduce a coefficient C_d and get the final total discharge formula as

$$Q = C_d \cdot H(1 - m) \cdot L\sqrt{2gmH}$$

$$= C_d \cdot \sqrt{2g} \cdot L \cdot H \cdot (1 - m) \cdot \sqrt{m} \cdot \sqrt{H}$$

$$= C_d \sqrt{2g}\, \sqrt{m}\, (1 - m) \cdot \sqrt{m} \cdot L \cdot H^{3/2}$$

or $$Q = C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2} \cdot [m^{1/2} - m^{3/2}] \qquad ...(iii)$$

The depth BC adjusts itself in such a way that the discharge passing through it, is maximum. Hence,

$$\frac{dQ}{dm}=0$$

Differentiating (*iii*), we get

$$\frac{dQ}{dm}\, C_d \cdot \sqrt{2g} \cdot L \cdot H^{3/2}\left[\tfrac{1}{2}m^{-1/2}-\tfrac{3}{2}m^{1/2}\right]=0$$

$$\therefore \quad \frac{1}{2}m^{-\frac{1}{2}}-\frac{3}{2}m^{\frac{1}{2}}=0$$

or $$m^{-\frac{1}{2}}=3m^{\frac{1}{2}}$$

or $$m=\tfrac{1}{3}.$$

Hence,
$$Q=C_d\cdot\sqrt{2g}\cdot L\cdot H^{3/2}\cdot\left[(\tfrac{1}{3})^{1/2}-(\tfrac{1}{3})^{3/2}\right]$$
$$=C_d\cdot\sqrt{2\times 9.81}\left[0.574-0.189\right]L\cdot H^{3/2}.$$
$$=1.71\, C_d\cdot L\cdot H^{3/2}$$

or
$$Q=1.71\cdot C_d\cdot L\cdot\left[y_u+\frac{V_a^2}{2g}\right]^{3/2} \qquad ...(15.12)$$

This is the required equation (15.12).

Derivation of orifice formula applied to bridge openings

With reference to Fig. 15.6.

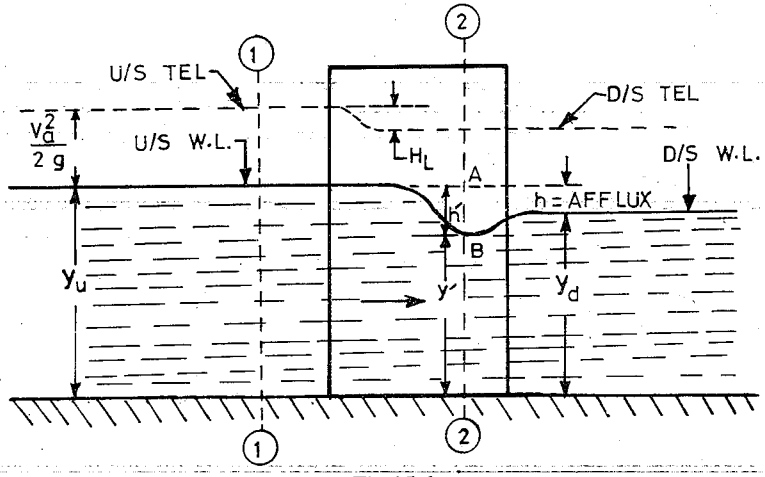


Fig 15.6

Applying Bernoulli's Equation at sections 1-1 and 2-2 and ignoring the loss (H_L) due to entry and friction, we get

$$y_u+\frac{V_a^2}{2g}=BC+\frac{V^2}{2g}=y'+\frac{V^2}{2g}$$

or
$$\frac{V^2}{2g}=\left[y_u-y'+\frac{V_a^2}{2g}\right]$$

or $$V = \sqrt{2g\left(y_u - y' + \frac{V_a^2}{2g}\right)}$$

Putting $y_u - y' = h'$, we get

$$V = \sqrt{2g\left(h' + \frac{V_a^2}{2g}\right)} \qquad ...(i)$$

The discharge through section 2-2 is given by

$$Q = \text{Area} \times \text{Velocity}$$

$$= (L \cdot y') \times \sqrt{2g\left(h' + \frac{V_a^2}{2g}\right)}$$

When afflux is small $\left(i.e. < \frac{y_d}{4}\right)$, y' is nearly equal to y_d, then

$$Q = L \cdot y_d \sqrt{2g} \cdot \left[h' + \frac{V_a^2}{2g}\right]^{1/2} \qquad ...(ii)$$

Now h is less than h' because when the flow emerges from the bridge, the water surface rises due to recovery of some velocity head as depth. If $e\left(\frac{V_a^2}{2g}\right)$ is the velocity head that is converted into potential head, we have

$$h' = h + e\left(\frac{V_a^2}{2g}\right) \qquad ...(iii)$$

Substituting in (*ii*), we get

$$Q = L \cdot y_d \cdot \sqrt{2g} \cdot \left[h + e \cdot \frac{V_a^2}{2g} + \frac{V_a^2}{2g}\right]^{1/2}$$

$$Q = L \cdot y_d \cdot \sqrt{2g} \cdot \left[h + (1+e)\frac{V_a^2}{2g}\right]^{1/2} \qquad i.e.\ \text{Eq. (15.13)}$$

This is the required equation (15.13).

(3) **By Moles-worth Formula.** The afflux for non-erodible beds may be easily computed approximately by using this empirical formula, as given below :

$$h = \left[\frac{V^2}{17.88} + 0.01524\right]\left[\left(\frac{A}{a}\right)^2 - 1\right] \qquad ...(15.14)$$

where h = afflux in metres

V = velocity in unobstructed stream in m/sec.

A = unobstructed sectional area of the stream in sq. m.

a = obstructed reduced sectional area of the stream in sq. m. at the bridge site

15.11. Structural Design and Other Detailing of Slab Bridges

While proceeding to design a small bridge across a drain or a canal, the design discharge, the total linear waterway, the span length, the floor level, the invert level of the bridge deck (after making a provision of clearance above H.F.L.), the normal and maximum scour depth, the depth of bridge foundations, etc. are, first of all, decided.

The width of road bridge and number of lanes which it is going to accommodate are then decided depending upon the importance of the road.

Generally, road bridges are designed for two lanes of class *A* or one lane of class *70 R* loading, whichever gives maximum reactions or stresses. The design procedure used for designing R.C.C. slab bridges or any other type of bridges is quite laborious and cumbersome, and the same is available in any good book of R.C.C. designs.

In actual practice, however, such detailed calculations for design of bridge decks, are usually avoided, particularly for small bridges and culverts, by making use of the standard deck designs of different spans, as published by the Ministry of Surface Transport (MOST) and Indian Road Congress (IRC).

These readymade designs not only provide the design of bridge deck, but also provide the maximum reactions that are likely to come on the piers or abutments, carrying such super structures. These reactions coupled with other forces can then be used for designing the piers as well as the abutments of such bridges.

The various forces acting on piers and abutments are taken in accordance with the provisions of I.R.C. publication No. 6–1966 titled "*Standard Specifications and Code of Practice for Road Bridges, Section II (Loads and Stresses)*.

The important forces which are taken in the design are, however, summarised below:

Forces on Piers :

(*i*) *Dead Load* (D.L.) from super-structure (available from readymade design tables).

(*ii*) *Live Load* (L.L.) from super-structure (available from readymade design tables).

(*iii*) *Impact Load** (I.L.) from super-structure (available from readymade design tables)

(*iv*) *Self weight of pier.*

(*v*) (*a*) *Longitudinal braking force, (P)* equal to 20% of first train load plus ten per cent of the load of the succeeding trains (in class 70 *R* loading, this is 70 tonnes). Thus, for ordinary two lane bridges, $P = 0.2 \times 70 = 14$ tonnes. (*b*) For multi-lane bridges, as in (*a*) above for first two lanes, plus five per cent of the loads in lanes in excess of two. Thus, for four lane bridges,

$$P = 0.3 \times 70\, t = 21 \text{ tonnes.}$$

(*vi*) The *wind load* and seismic forces (either of the two) should also be considered wherever predominant. Since the consideration of these forces permit to increase the permissible stresses in brick or concrete (materials used for piers) by 25% or so ; for ordinary designs in non-seismic or less seismic regions and where bridges are not at much height above the natural ground levels, these forces may often be neglected.

(*vii*) The *forces caused due to buoyancy of water* (*i.e.* reduction in the weight of pier material due to uplift exerted by the water surrounding the pier) may also be considered in design. Often this is also neglected when pier thickness, etc. are governed by the minimum permissible limits.

* The 50% impact load is considered to be a part of live load for calculating pressure at the bottom surface of the bed block ; and an impact load varying between 50% to zero is considered for calculating the pressure on the top 3 m of structure below the bed block. Thus, no impact is considered for examining pier or abutment sections below 3 m from bed block.

(*viii*) *Other minor forces* such as horizontal forces caused by water currents, etc.

Forces on Abutments. Besides D.L., L.L., self weight, seismic or wind forces and buoyancy if there, the important force which comes into play is the pressure exerted by the soil fill from behind the abutment. The soil may be dry or wet or saturated depending upon the site conditions. The horizontal braking force and the horizontal force caused by temperature variations will no doubt act in addition to the soil pressure. All these forces may be considered and abutment designed. No tension should be permitted in masonry piers, and the base should be wide enough so that the soil pressure does not exceed the permissible bearing capacity of the soil.

The detailed designs of bridges are beyond the scope of this book, and the students may refer to some standard book on bridge designs.

CAUSEWAYS AND BOX CULVERTS

15.12. Causeways

When a road embankment is constructed across a flat terrain, the runoff from the area having no deep and defined channels in it, may have to be disposed of by some suitable means. In such cases, the surface water will go on collecting on one side of the road embankment, unless drained out either through culvert openings inside the embankment or by allowing the water to submerge the dipped road at frequent intervals. *The second arrangement called a causeway consists in lowering or dipping the road to the ground level at frequent intervals, and thus allowing the water to flow across the road.* The provision of such a dip or causeway in the longitudinal profile of the road and letting the drainage water pass over them, may be feasible in less important towns, and may generally be impracticable because of various limitations, such as : (*i*) flow of water over numerous sections of the road makes its proper maintenance difficult and expensive; (*ii*) when the embankment is constructed high above the ground (which may be necessitated in swampy and water-logged areas), the provision of causeways will require dipping down the high road levels to the ground at frequent intervals and will produce a very undesirable road profile. Hence, under all such circumstances, it may be better to construct culverts at intervals rather than frequently dipping the road for crossing the surface water.

15.13. Pipe Culverts Flowing Full and Box Culverts

After having decided to construct a culvert across some such flat terrain, the runoff likely to be passed may be calculated. Now, the water coming towards the road in such cases is having very low velocities and cannot be passed below the road at those velocities, because in that case, the area of the opening will be very large. Hence, in such cases, the design is based on an increased velocity of flow through the culvert, and to create that velocity, the water is allowed to head up at the inlet end of the culvert. The greater is the heading up of water at the inlet end, the greater is the head causing flow ; and hence, the greater will be the flow velocity and lesser the opening. But this heading up of water can be allowed only to a safe predetermined level, and should be fixed, so that the road bank is not overtopped nor any property in the flood plain damaged.

After fixing the upstream level up to which the water will head up while passing maximum design discharge, the downstream water level is determined. This level will

depend on the hydraulic conditions of the tail channel, if at all existing, and is generally taken as equal to the surface level of the natural unobstructed flow at the site before the road embankment is constructed.

The operating head, i.e. the difference of the upstream and downstream levels (say H_L) will give us the maximum head causing flow. The area of the opening should then be decided, so that it is sufficient to pass the design discharge.

If V is the velocity through the culvert opening, which runs full in such a case, then the head loss (H_L) will be equal to the sum of the entrance loss, friction loss in the barrel, and the velocity head in the barrel. The entrance loss depends on the type of entrance provided, and may be taken equal to $0.505 \frac{V^2}{2g}$ for a square edged entrance, and equal to $0.05 \frac{V^2}{2g}$ for a well rounded

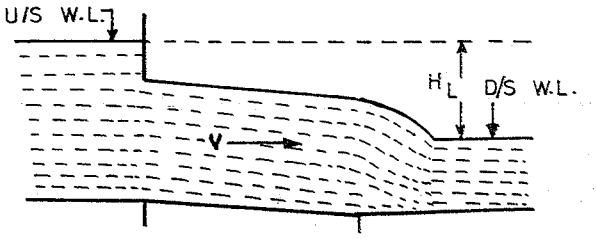


Fig. 15.7. Box or pipe culvert running full even when the outlet is not submerged.

entrance. The friction loss may be calculated by Manning's formula as $\frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$

Hence, H_L = Entrance loss + Friction loss + Velocity head in barrel

$$= K_e \frac{V^2}{2g} + \frac{n^2 \cdot L \cdot V^2}{R^{4/3}} + \frac{V^2}{2g}$$

where $K_e = 0.505$ for square edged entrance

$= 0.05$ for bell mouthed entrance.

$$\therefore \quad H_L = K_e \cdot \frac{V^2}{2g} + \frac{V^2}{2g} + \frac{n^2 \cdot L}{R^{4/3}} \cdot 2g \left(\frac{V^2}{2g}\right)$$

$$= \frac{V^2}{2g}\left[1 + K_e + \frac{n^2 \cdot L \cdot 2g}{R^{4/3}}\right] = \frac{V^2}{2g}\left[1 + K_e + K_f\right]$$

where $K_f = \frac{n^2 \cdot L \cdot 2g}{R^{4/3}}$

or $$V = \sqrt{\frac{2g \cdot H_L}{1 + K_e + K_f}} \; ; \; \therefore \; A \cdot V = A \cdot \sqrt{\frac{2g\, H_L}{1 + K_e + K_f}}.$$

Hence, knowing the area, the discharging capacity of the culvert, or by knowing the discharge, the required area can be easily computed. The required sized pipe (or a rectangular barrel) may then be constructed through the road enbankment.

PROBLEMS

1. Discuss briefly the various principles and guidelines which help us in fixing the hydraulic design of a bridge culvert.

2. (*a*) Distinguish between 'normal scour depth' and 'maximum scour depth' in connection with bridge design. How does it help in determining the depth of bridge foundation ?

(*b*) How will you fix up the number of spans for a bridge after computing its required linear waterway ?

(*c*) "Reducing the waterway of a bridge will reduce the cost of superstructure, but will increase the cost of sub-structure." Discuss critically the above statement.

3. Write short notes on any two of the following :

(*i*) Afflux at bridges.

(*ii*) Waterway calculations for bridges.

(*iii*) Waterway and scour depth provision in bridge design.

(*iv*) Economical span length for bridges.

(*v*) Provision of cause-ways in road construction.

(*vi*) Box culverts and their afflux computations.

16

Ground Water Hydrology and Construction of Wells and Tubewells

16.1. Definition and General Introduction

Ground water is the underground water that occurs in the saturated zone of variable thickness and depth, below the Earth's surface. Cracks and pores in the existing rocks and unconsolidated crystal layers, make up a large underground reservoir, where part of precipitation is stored.

Ground-water is largely tapped for irrigating crops in India. So much so that about 46% of our total irrigated area, gets its irrigation water from this source. Most of our **minor irrigation schemes***, make use of this source of supply. Besides its use for irrigation, ground water is also used as a source of water supply for municipal purposes.

The ground water is utilised through wells and tubewells. Various lifting devices, such as those using animal, manual, diesel, or electric power, may be used, so as to bring the underground supplies to the surface.

The use of open wells is a traditional method of tapping ground water in areas where ground watertable is high. Manual, animal, wind, diesel or electric power can be used for lifting water from open wells. The use of tubewells, however, is a subsequent development in the techniques of tapping ground water, and certainly requires diesel or electric power.

16.2. Occurrence of Ground Water

The rainfall that percolates below the ground surface, passes through the voids of the rocks, and joins the watertable. These voids are generally inter-connected, permitting the movement of the ground-water. But in some rocks, they may be isolated, and thus, preventing the movement of water between the interstices. The mode of occurrence of ground-water therefore, depends largely upon the type of formation, and hence depends upon the geology of the area.

The possibility of occurrence of ground-water mainly depends upon two geological factors; *i.e.* (*i*) the *porosity* of the rocks; and (*ii*) *the permeability* of the rocks, as explained below:

16.2.1. Porosity. The porosity of a rock, which is the major geological criteria for occurrence of ground-water, is a quantitative measurement of the interstices or voids present in the rock. It is generally defined as *the percentage of the voids present in a given volume of aggregate*. Mathematically, it can be expressed as :

* Schemes involving CCA up to 2000 hectares are **minor irrigation schemes**; those involving CCA between 2000 to 10,000 hectares are **medium irrigation schemes**; and those involving CCA greater than 10,000 hectares are **major irrigation schemes.**

$$\text{Porosity} = \frac{\text{Total volume of voids in the aggregate, } i.e. \text{ the volume of water required to saturate the dry sample } (V_v)}{\text{Total volume of the aggregate } (V)}$$

It is generally denoted by the letter n.

$$\therefore \qquad n = \frac{V_v}{V} \times 100 \text{ (per cent)} \qquad \text{...(16.1)}$$

Porosity, in fact depends upon the shape, packing, and degree of sorting of the component grains in a given material. Uniform and well sorted grains (Fig. 16.1 A) give rise to higher porosity; whereas, heterogeneous grains with irregular arrangement (Fig. 16.1B) decrease the porosity.

The porosity of rocks and unconsolidated materials may vary considerably. It may be less than a per cent or more than 50%. But generally, it does not exceed 40% except in very poorly compacted materials. In general, a porosity greater than 20% is considered to be large, and below 5% as small, and between 5 to 20% as medium. The porosity values for a few common types of rock formations are given in table 16.1.

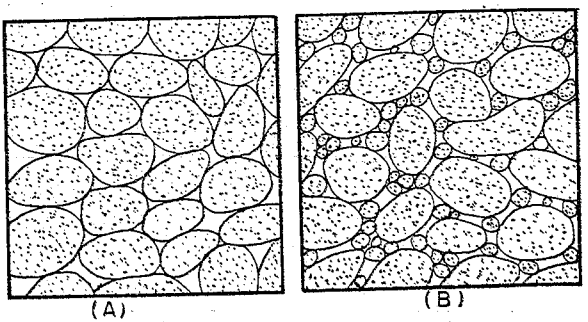


Fig. 16.1. Diagrammatic sketch of well sorted (*A*) and poorly sorted (*B*) deposits. Porosity of *A* is greater than that of *B*, because the grains of *A* are all about equal in size. In *B*, smaller grains fill the spaces between the larger grains, thus reducing the volume of void space.

Table 16.1. Porosity values of a few rock formations

S. No.	*Type of rock formation*	*Porosity*
1.	Granite, Quartzite	1.5%
2.	Slate, Shale	4%
3.	Limestone	5 to 10%
4.	Sandstone	10 to 15%
5.	Sand and Gravel	20 to 30%
6.	Only Gravel	25%
7.	Only Sand	35%
8.	Clay and Soil	45%

16.2.2. Permeability and Transmissibility. As stated above, the ground water can get stored in the underground rocks, only if, they are sufficiently porous. In other words, water is get stored in the pores (voids) of the rocks. The *porosity* of the rock, thus, defining the maximum amount of water that can be stored in the rock. The porosity, however, in itself, does not ensure the storage of underground water. Infact the water can enter into a rock (with any amount of porosity) only if the rock permits the flow of water through it, *i.e.* it depends on whether the rock is *permeable* or not. It may be clarified here that a rock which is porous, may or may not be permeable. For

example, Shale is a porous rock, but its pore spaces are so minute that the rock remains impermeable. The *size of the pores*, is thus, quite an important factor, and it should be sufficiently large to make the rock permeable.

The *permeability* is, therefore, defined as the ability of a rock or unconsolidated sediment, to transmit or pass water through itself. Transmissibility is another term which represents the same physical meaning, but only differing mathematically, as explained below :

The capability of the entire soil of full width (b) and depth (d), (*i.e.* area bd) is represented by *permeability*; while that of the soil of unit width and full depth (*i.e.* $b = 1$ and $d = d$, *i.e.* $A = d$) is known as *transmissibility.*

The permeability is measured in terms of **coefficient of permeability** which will be defined a little later, in article 16.6. Various methods including constant head permeameter and variable head permeameter, are used to measure permeability.

16.3. Zones of Under-ground Water

As we move down below the surface of the Earth towards its centre, water is found to exist in different forms in different regions.

With regard to the existence of water at different depths, the Earth's crust can be divided into various zones, namely,

(*i*) Zone of rock fracture; and

(*ii*) Zone of rock flowage.

The depth of the *zone of rock flowage* (*i.e.* the zone in which the rocks undergo permanent deformation) is not accurately known but is generally estimated as many miles. Interstices are probably absent in this zone, because the stresses are beyond the elastic limits and the rocks remain in a state of plastic flow. Water present in this zone is known as *internal water*, and a hydraulic engineer has nothing to do with this water.

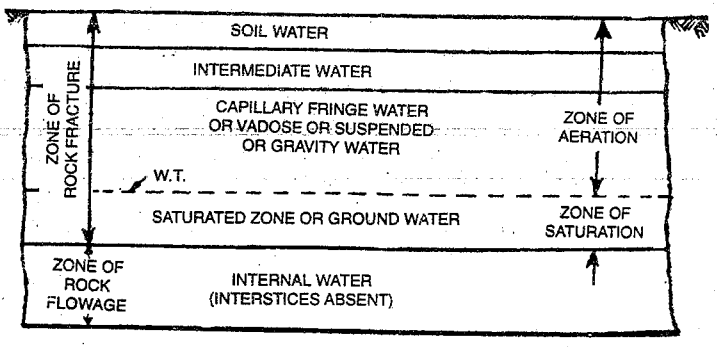


Fig. 16.2. Zones of under-ground water.

Above the zone of rock flowage, there lies the *zone of rock fracture*. In this zone, the stresses are within the elastic limits, and the interstices do exist. Water is stored in the voids, the amount of which depends upon porosity. The maximum depth of this zone below the ground surface, varies in the range of about 100 metres or less to 1,000 metres or more.

In crystalline rocks, most of the water is met within 100m of the surface; while in sedimentary rocks, it is found upto depths of 1,800 m or so, although very less quantity is found below 1,000 m or so.

The zone of rock fracture can be further sub divided into two zones; one is the *zone of saturation*, *i.e.* the zone below the watertable, and the other is the *zone of aeration*, *i.e.* the zone above the watertable.

In the *zone of saturation*, water exists within the interstices, and is known as ground water. This is the most important zone for a ground water hydraulic engineer, because he has to tap out this water. Water in this zone is under hydrostatic pressure.

The space above the watertable and below the surface is known as the *zone of aeration*. Water exists in this zone by molecular attraction. The gravity water moves through this zone, and the water in this zone is not at hydrostatic pressure. The thickness of this zone varies from almost none in marshy and low lying areas to about 300 metres or so in arid regions.

This zone is also divided into three classes depending upon the number of interstices present. The *capillary fringe* is the belt overlying the zone of saturation and it does contain some interstitial water, and is thus a continuation to the zone of saturation; while the depth from the surface which is penetrated by the roots of vegetation is known as the *soil zone*. The remainder intermediate part is the *intermediate zone.*

These zones have already been explained in article 2.14 and the students may refer back.

16.4. Movement of Ground Water and its Velocity

16.4.1. The Watertable. The static level of water in wells penetrating the zone of saturation, is called the watertable. The watertable is often described as the subdued replica of the surface topography. It is generally higher under the hills and lower under the valleys, and a contour map of the watertable in any area may look like the surface topography.

The watertable is thus the surface of a water body which is constantly adjusting itself towards an equilibrium condition, with the water moving from the higher points to the lower points. If there were no recharge to or outflow from the ground water in a basin, the watertable would eventually become horizontal. But, few basins have uniform recharge conditions at the surface, as some areas receive more rain than others; and some portions of the basin have more permeable soil. Thus, when intermittent recharge does occur, mounds and ridges do form in the watertable under the areas of greatest recharge. Subsequent recharge creates additional mounds, perhaps at other points in the basin, and the flow patten is further changed. Various other factors, such as : variations in permeability of aquifers; impermeable strata ; influence of lakes, streams, and wells, etc; do make the watertable less and less horizontal. *All this gives us a picture of watertable constantly adjusting towards equilibrium* (*i.e.* horizontal). Because of the low flow rates in most of the aquifers, this equilibrium is rarely attained before additional disturbances occur.

16.4.2. Movement of Ground Water. As discussed above, the watertable is generally not horizontal, and has high and low points in it, *i.e.* it is not in equilibrium. In order that the equilibrium is approached, water moves inside the ground from the high points on the watertable to the points lower down, as shown in Fig. 16.3. The rate at which such movement occurs is dependent upon two factors, *i.e.* (*i*) on the ability of the porous medium to pass water through it, *i.e.* on the *permeability*; and (*ii*) on the

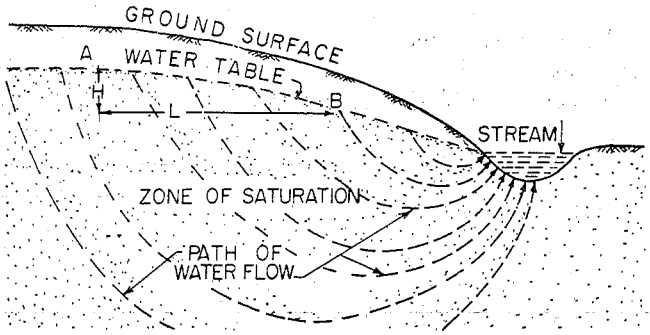


Fig. 16.3. Cross-sectional diagram showing the flow of ground water in a uniformly permeable material. The difference in elevation between point *A* and *B* of the water-table is *H* and the distance between them in *L*. The hydraulic gradient is *H/L*.

driving force or *hydraulic gradient* (*I*) usually expressed as the ratio between the difference in elevation (H_L) of the two points on the watertable (in the direction of flow), and the distance between then (*L*). This was stated as such by Darcy, as the important **Darcy's law**, which is discussed below :

16.4.3. Darcy's Law for determining Ground Water Velocity. On the basis of experimental evidence, Mr. H. Darcy, a French Scientist enunciated in 1865, a law governing the rate of flow (*i.e.* the discharge) through soils. According to him, this discharge was directly proportional to the head loss (H_L), and the area of cross- section (*A*) of the soil, and inversely proportional to the length of the soil sample (*L*). In other words,

$$Q \propto \frac{H_L}{L} \cdot A$$

But $\frac{H_L}{L}$ represents the rate of loss of head, *i.e.* the hydraulic gradient (*I*)

$$\therefore \quad Q \propto I \cdot A$$

or
$$Q = K \cdot I \cdot A \qquad \ldots(16.2)$$

where, *K* is the proportionality constant and was found to be changing with the type of soil, and hence represented a property of the soil, called *permeability* or *coefficient of permeability.*

The above equation becomes dimensionally compatible, if *K* has the units of *L/T*, *i.e.* say cm/sec, *i.e.* the units of velocity.

The Darcy's law has been demonstrated to be valid only for *laminar** flow conditions, which as far as soils are concerned, is not at all a serious problem, because the flow in sands, silts and clays is invariably laminar.

Dividing both sides of equation (16.2) by *A*, we get

$$\frac{Q}{A} = K \cdot I$$

or
$$v = K \cdot I \qquad \ldots(16.2\ a)$$

* For laminar flow and turbulent flow differences, please refer to any book on Fluid Mechanics.

where v is the discharge velocity, and is not the actual flow velocity through the soil medium, since the flow occurs through the voids of cross-sectional area A_v and not in A itself. The permeability of the soil can then be viewed as this superficial velocity under a unit hydraulic gradient.

If A_v is the area of the voids, then $A_v \cdot v_a = A \cdot v$, where v_a is the actual velocity of flow of water through the soil. Then

$$v = v_a \cdot \frac{A_v}{A} \qquad ...(16.3)$$

when A is large in comparison, we can safely assume that the ratio of the area of the void (A_v) to the total area (A) is the same as the ratio of the volume of the voids (V_v) to the total volume (V), *i.e.* equal to porosity (n).

Hence, $\frac{A_v}{A} = n$ (porosity)

Substituting this value is eqn. (16.3), we get

or $$\boxed{v = n \cdot v_a} \qquad ...(16.5)$$

Knowing the value of v from equation (16.2*a*) and dividing it by porosity n, the actual velocity of flow (v_a) of water through the soil, can be worked out.

16.4.4. Empirical Formulas for Ground Water Velocity Determination. Before Darcy came into picture, certain empirical formulae based upon the experimental results were the only way to find out the velocity of ground water flow. The formulae which were commonly used, are :

(1) **Slichter's formula.** According to which

$$v_a = K' I \frac{D^2_{10}}{\mu} \qquad ...(16.5)$$

where v_a = velocity of ground water flow in m/day.
K' = a constant.
I = slope of the hydraulic gradient line.
D_{10} = effective size of the particles in the aquifer in mm. (*i.e.* the hypothetical size which is larger than 10% of the particles in the sample, *i.e.* only 10% of the particles will pass through this size).
μ = viscosity of water depending on temperature.

(2) **Hazen's formula.** Formula in M.K.S. or S.I. system is

$$v_a = \frac{K'' I D^2_{10}}{60} \times (1.8T \div 42) \qquad ...(16.6)$$

where v_a = velocity of ground water flow in m/day.
T = temperature in °C.

values of K' and K'' in M.K.S. or S.I. system are approximately 400 and 1000, respectively.

Example 16.1. *Find out the velocities of the ground water flow with the following data, using Slichter and Hazen's constants as 400 and 800, respectively.*

Viscosity coefficient of water at ground water temperature of 10° C = *1*
Effective size of the particles in the aquifer = *0.1 mm*
Hydraulic gradient = *1 in 80*

Solution. (*a*) Using Slichter's formula *i.e.* eqn. (16.5), we have

$$v_a = \frac{K' I \cdot D_{10}^2}{\mu}$$

$$= \frac{400 \times \frac{1}{80} \times (0.1)^2}{1} = \frac{400 \times 0.01}{80} = \mathbf{0.05\ m/day} \quad \textbf{Ans.}$$

(b) Using Hazen's formula, i.e. Eq. (16.6), we have

$$v_a = \frac{K'' I \cdot D_{10}^2}{60}(1.8T + 42), \text{ where } T \text{ is in °C}$$

$$= \frac{800 \times \frac{1}{80} \times 0.01}{60}(1.8 \times 10 + 42) \qquad [\because T = 10°C]$$

$$= \frac{800 \times \frac{1}{80} \times 0.01 \times 60}{60} = \mathbf{0.1\ m/day} \quad \textbf{Ans.}$$

16.4.5. Permeability Values and Relation between *K* and *T*. From Darcy's law, the coefficient of permeability may be defined as the rate of flow of water through a unit cross sectional area of the water- bearing material under a unit hydraulic gradient, and at a temperature* of 20°C. Various approximate average values of permeability coefficient for different types of rock (soil) formations are given in table 16.2. There may be vide variations in these values, depending upon the type of field soil.

Table 16.2. Permeability Coefficient (K) Values

S. No.	*Type of rock formation*	*App. Av. value of K in cm/sec*
1.	Granite, Quartzite	0.6×10^{-5}
2.	Slate, Shale	4×10^{-5}
3.	Limestone	4×10^{-5}
4.	Sandstone	0.004
5.	Sand and Gravel	0.4
6.	Only Gravel	4.0
7.	Only Sand	0.04
8.	Clay and Soil	0.04×10^{-5}

* This accounts for the viscosity of water, which decreases with the increase in temperature, thereby causing permeability to increase with the increase in temperature.

The term *transmissibility* introduced by Theiss is measured by the **coefficient of transmissibility (T),** which is defined as the rate of flow of water through a vertical strip of the water bearing material (*i.e.* aquifer) of unit width and full depth (d), under a unit hydraulic gradient and at a temperature of 20°C.

The relation between K and T is simple, and is given by

$$T = Kd \qquad ...(16.7)$$

16.5. Drainage of Ground Water

By the term 'drainage of ground water', we generally mean extracting the water from below the watertable through wells, infiltration galleries, springs, etc. The water, is thus, drained from the ground-water reservoir, either under a natural phenomenon (like spring), or it can be drained artificially by constructing wells, and lifting water through them. The water so drained may then be used to fulfil irrigation or municipal needs.

16.6. Ground Water Yield

The interstices present in the given formation get filled up with water during the process of ground-water replenishment. If all these voids are completely filled with water, then it is known as a saturated formation. The water contained in these voids is drained by digging wells under the action of gravity drainage. When these saturated formations are drained under the action of gravity drainage, it is found that the volume of water so drained is less than the volume of the void space, as indicated by its porosity. This is because of the fact, that the entire water contained in these voids cannot be drained out by the mere force of gravity. Some of the water is always retained by these interstices due to their molecular attraction. The water so retained is known as **pellicular water.**

16.6.1. Specific Yield. The volume of ground water extracted by gravity-drainage from a saturated water bearing material is known as the **yield**, and when it is expressed as ratio of the volume of the total material drained, then it known as the *specific yield.*

$$\therefore \text{Specific yield} = \frac{\text{Volume of the water obtained by gravity drainage}}{\text{total volume of the material drained or dewatered}} \times 100 \qquad ...(16.8)$$

16.6.2. Specific Retention or Field Capacity. On the other hand, the quantity of water retained by the material against the pull of gravity is termed as the *specific retention* or the *field capacity;* and this is also expressed as the percentage of the total volume of the material drained.

Specific retention or field capacity

$$= \frac{\text{Volume of the water held against gravity drainage}}{\text{total volume of the material drained}} \times 100 \qquad ...(16.9)$$

It is evident that the sum of the specific yield and the specific retention is equal to its porosity.

16.6.3. Specific Retention of Different Kinds of Formations. As has been said earlier, the specific retention is the amount of the water held between the grains due to molecular attraction. This film of water is thus held by molecular adhesion on the walls of the interstices. Therefore, the amount of this water will depend upon the total

interstitial surface in the rock. If the total interstitial surface is more, the specific retention will be more, and *vice versa.*

Now, if the effective size of the soil grains decreases, the surface area between the interstices will increase, thus, causing more specific retention and less specific yield.

It, therefore, follows that, in fine soils like clay, the specific retention would be more, and hence, it would result in very small specific yield.

The reverse is also true when the grain size increases, the interstitial surface area reduces, and therefore, the specific retention reduces, and hence specific yield increases. It, therefore, follows that in a large particle soil like *coarse gravel*, the specific retention would be small, and it would result in large specific yield.

This conclusion is very important from practical stand point because it follows from this *that a water bearing formation of coarse gravel would supply large quantities of water to wells, whereas the clay formations although saturated and of high porosity would be of little value in this respect.* Hence, the location of wells depends considerably upon the type of neighbouring formations.

16.6.4. Determination of Specific yield of a Formation. It is difficult to determine accurately the specific yield in the laboratory, because it is difficult to obtain an undisturbed sample, and also the short-sample columns used in the laboratory cannot duplicate the very long capillary tubes actually existing in the field. And hence, the field observations are made through the medium of pumping tests at site, in order to determine accurate and reliable values of specific yields, as will be explained later.

16.7. Aquifers and their Types

A permeable stratum or a geological formation of permeable material, which is capable of yielding appreciable quantities of ground water under gravity, is known, as an *aquifer.* The term 'appreciable quantity' is relative, depending upon the availability of ground-water. In the regions, where ground-water is available with great difficulty, even fine-grained materials containing very less quantities of water may be classified as the principal aquifers.

When an aquifer is overlain by a confined bed of impervious material, then this confined bed of overburden is called an **aquiclude**, as shown in Fig. 16.4.

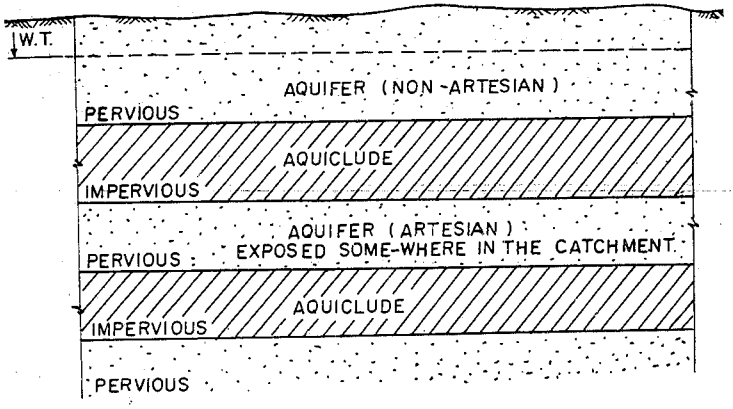


Fig. 16.4. Aquifers and aquicludes.

The amount of water yielded by a well, excavated through an aquifer, depends on many factors; some of which, such as the well diameter are inherent in the well itself. But all other things being equal, the permeability and the thickness of the aquifer are the most important.

Aquifers vary in depth, lateral extent, and thickness; but in general, all aquifers fall into one of the two categories, *i.e.*

1. Unconfined or Non-artesian aquifers, and
2. Confined or Artesian aquifers.

16.7.1. Unconfined Aquifers or Non-artesian Aquifers. The top most water bearing stratum, having no confined impermeable over burden (*i.e.* aquiclude) lying over it, is known as an *unconfined* aquifer or *non-artesian* aquifer. (Refer Fig. 16.5).

The ordinary gravity wells of 2 to 5 m diameter, which are constructed to tap water from the top most water bearing strata, *i.e.* from the unconfined aquifers,. are known as *unconfined* or *non-artesian* wells. The water level in these wells will be equal to the level of the watertable. Such wells are, therefore, also known as *wells* or *gravity wells.*

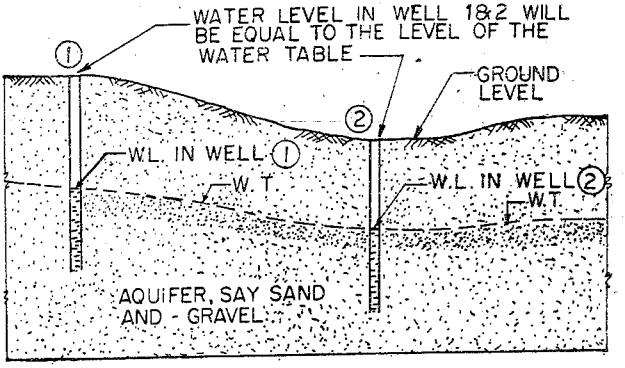


Fig. 16.5. Non-artesian or Unconfined aquifers and wells.

16.7.2. Confined Aquifers or Artesian Aquifers. When an aquifer is confined on its upper and under surface by impervious rock formations (*i.e.* aquicludes), and is also broadly inclined so as to expose the aquifer somewhere to the catchment area at a higher level for the creation of sufficient hydraulic head, it is called a *confined aquifer* or an *artesian aquifer*. A well excavated through such an aquifer, yields water that often flows out automatically, under the hydrostatic pressure, and may thus, even rise or gush out of surface for a reasonable height. However, where the ground profile is high, the water may remain well below the ground level. The former type of artesian well, where water gushes out automatically, is called a **flowing well.**

The level to which the water will rise in an artesian well is determined by the highest point on the aquifer, from where it is fed from the rain falling in the catchment. However, the water will not rise to this full height, because the friction of the water moving through the aquifer uses up some of the energy.

The question whether it will be a *flowing artesian well* or a *non-flowing artesian well* depends upon the topography of the area, and is not the inherent property of the artesian aquifer. In fact, if the *pressure surface** lies above the ground surface, the well will be a flowing artesian well; whereas, if the pressure surface is below the ground

* It is the imaginary surface which represents the magnitude of the hydrostatic pressure available along the artesian aquifer. It is represented by the line joining the various piezometric heads in various tightly cased wells tapping the aquifer.

surface, the well will be artesian but non- flowing, and will require a pump to bring the water to the surface, as shown in Fig. 16.6. Such non-flowing artesian wells are sometimes called as sub-artesian wells.

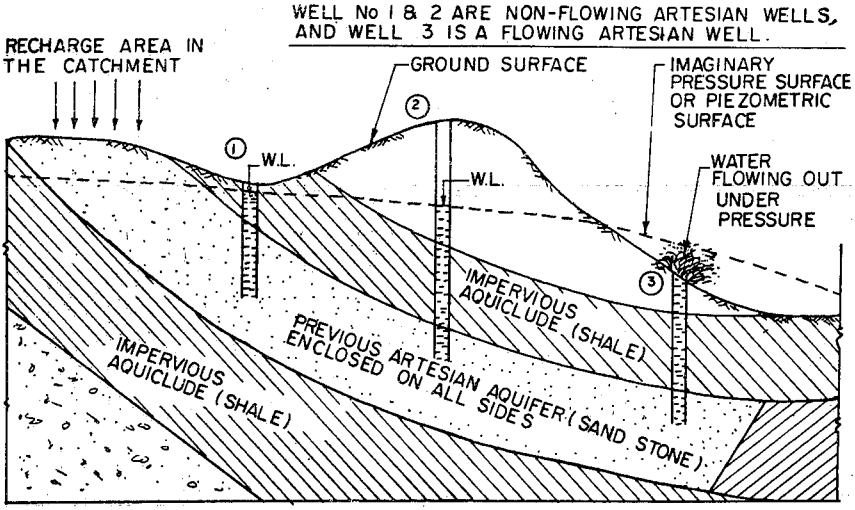


Fig. 16.6. Confined or Artesian aquifers and wells.

16.7.3. Perched Aquifers. Perched aquifer is a special case which is sometimes found to occur within an unconfined aquifer.

If within the zone of saturation, an impervious deposit below a pervious deposit is found to support a body of the saturated material, then this body of the saturated material, which is a kind of aquifer, is known as a *perched aquifer*. The top surface of the water held in the perched aquifer is known as the *perched watertable*. This is shown in Fig. 16.7.

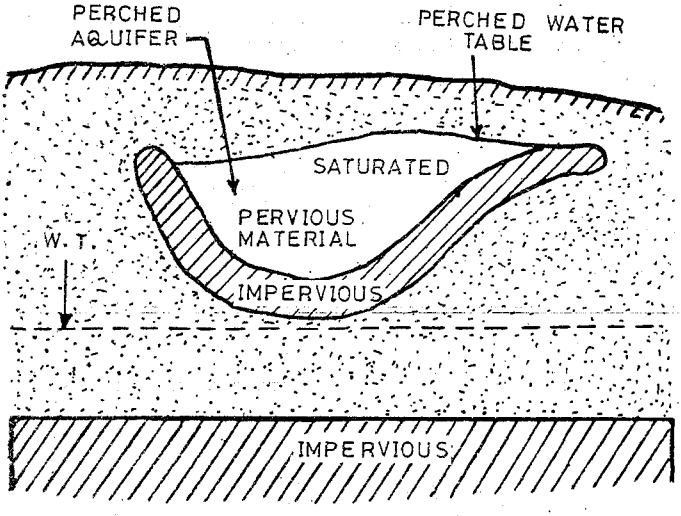


Fig. 16.7. Perched aquifer.

16.8. Certain Other Important Terms Connected with Ground Water

16.8.1. Specific Capacity. Specific capacity of a well is the rate of flow from a well per unit of drawdown. It should be determined for the fall of the first metre, as it is not the same for all the drawdowns.

16.8.2. Coefficient of Storage. Now, we introduce a very important term known as storage coefficient, which may be denoted by A.

Water discharged from an aquifer or recharged into an aquifer represents the change in its storage volume. For unconfined aquifers, this change can be easily determined. Knowing the fall or the rise of the watertable in a given time (t) and multiplying it with the average specific capacity during this time, the change of the storage volume in a time t can be found out.

But in artesian aquifers, (assuming that the aquifer itself remains saturated throughout the drainage), changes in pressure produce only small changes in storage volume. Thus, the hydrostatic pressure within an aquifer partially supports the overburden, while the remainder is supported by the solid structure of the aquifer, When the hydrostatic pressure is reduced, such as by pumping water from a well penetrating the aquifer, the aquifer load increases. The aquifer gets compressed and forces some water from it. In addition, lowering of the pressure causes a small expansion and subsequent release of water. This water yielding capacity of an artesian aquifer can be expressed by its storage coefficient.

The storage coefficient (A) for an artesian aquifer is equal to the volume of the water released from the aquifer of unit cross- sectional area and of the full height of the aquifer when the piezometric surface declines by unity. In general, storage coefficient is defined as the volume of water that an aquifer releases or stores per unit surface area of the aquifer per unit change in the component of head normal to that surface.

16.9. Measurement of Yield of Underground Sources (Aquifers)

The yield of an aquifer can be estimated in the following different ways ;

(*i*) *On the basis of flow velocity of the ground water;* and
(*ii*) *By performing pumping tests in the field.*

These methods are discussed below.

16.9.1. Estimation of the Yield by Estimation of the Velocity of Ground Water. If a well is penetrated through an aquifer, the water will rush into it with a velocity say v; and if A is the area of the aquifer opening into the well, then Q will be given as :

$$Q = v \cdot A \qquad ...(16.10)$$

where v is the discharge velocity into the well, and is given by the eqn. 16.4) as :

$$\boxed{v = n \cdot v_a}$$

where n = porosity of the soil medium
v_a = actual flow velocity of ground water.

Eventually, the discharge of the well excavated through the given aquifer is given as:

$$Q = n \cdot v_a \cdot A \qquad ...(16.11)$$

In the above equation, the velocity of the ground water flow (v_a) can be estimated by using Slichters or Hazen's empirical equations (eqn. 16.5 and 16.6, respectively); or it can better be measured in the actual field by using chemical tracers, such as a dye ; or by using electrical resistivity methods.

The time (t) taken by a **chemical tracer** to travel a given known distance (S) between two observation wells, will directly indicate the ground flow velocity as, $v_a = \frac{S}{t}$. Such a test can also help to determine K, since $v = K \cdot I$; or $K = \frac{v}{I} = \frac{n \cdot v_a}{I}$ where $I = \frac{H_L}{S}$ where H_L is the difference of water surface elevations of the two wells.

Example 16.2. *In a field test, a time of 6 hours was required for a tracer to travel through an aquifer from one well to another. The observation wells were 42m apart, and the difference in their water levels was found to be 0.42 m. Compute (i) the discharge velocity; (ii) the coefficient of permeability (K). Given the porosity of the soil medium as 20%. (iii) Also compute the value of coefficient of intrinsic permeability for the aquifer in Darcy's, if viscosity of water* $\nu = 0.01\ cm^2/s$.

Solution. Velocity of the ground water flow $= v_a = \frac{S}{t} = \frac{42\text{ m}}{6\text{ h}}$

$$= \frac{4200}{6 \times 60 \times 60}\ \text{cm/sec} = 0.194\ \text{cm/sec}$$

The discharge velocity $= v = n \cdot v_a = 0.2 \times 0.194\ \text{cm/sec} = \mathbf{0.0388\ cm/sec}$ **Ans.**

Hydraulic gradient between the wells $= I = \frac{H_L}{S} = \frac{0.42\text{ m}}{42\text{ m}} = \frac{1}{100}$

using : $v = K \cdot I$, we have $0.0388\ \text{cm/sec} = K \times \frac{1}{100}$

or $$K = 100 \times 0.0388\ \text{cm/sec} = 3.88\ \text{cm/sec}.$$

$\therefore$ coefficient of permeability $= \mathbf{3.88\ cm/sec}$ **Ans.**

Intrinsic permeability coefficient is given by the eqn.

$$K_0 = \frac{K \cdot \nu}{g} \qquad ...(16.12)$$

$$= \frac{3.88\ \text{cm/sec} \times 0.01\ \text{cm}^2/\text{sec}}{981\ \text{cm/sec}^2}$$

$$= \frac{3.88 \times 0.01}{981}\ \text{cm}^2 = 3.96 \times 10^{-5}\ \text{cm}^2$$

Since $\mathbf{9.87 \times 10^{-9}\ cm^2 = 1\ darcy}$, we have

K_0 in darcy $= \frac{3.96 \times 10^{-5}}{9.87 \times 10^{-9}}\ \text{darcys} = \mathbf{4007\ darcys}$ **Ans.**

16.9.2. Estimation of Yield by Pumping Tests. The yield of an aquifer can be estimated by conducting two direct practical tests in the field, which can directly indicate the well yield from the given aquifer. These tests are:

(*a*) Pumping test ; (*b*) Recuperating test

Both these tests are discussed below:

(*a*) **Pumping test.** A well is, first of all, constructed through the aquifer, of which the yield is to be estimated. Huge amount of water is drawn from the well, so as to cause heavy drawdown in its water level. The rate of pumping is changed and so adjusted that the water level in the well becomes constant. In this condition of equilibrium, the rate of pumping will be equal to the rate of yield, and hence, the rate of pumping will directly give us the yield of the well at a particular drawdown.

But it is very difficult and almost impractical to adjust the rate, so as to keep the well water level constant.

(*b*) **Recuperating test.** In this method, water is first of all drained from the well at a fast rate, so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The rise is noted at regular intervals of time, till the initial level is reached. Knowing the area of the well and the rise of the water level, the volume of the water yielded in that given time interval, can be worked out at different drawdowns.

It is found that the yield is higher at higher drawdowns. These tests are generally conducted during the driest periods of years, so as to know the yield under the worst conditions.

In addition to the above two direct practical tests for determining the yield of a well or an aquifer, another pumping test with one, two, or more observation wells can be performed to determine the characteristics of an aquifer. Although, dominant hydraulic properties of the aquifer can be computed even by the use of one observation well with a main pumping well, yet more precise results could be obtained by installing two or more observation wells at varying distances from the main central pumped well. To obtain precise & reliable results, the pumping in the main well may be continued for a long period of 15 to 20 hours or more, to obtain study conditions of water levels in the wells. However, it is not necessary to wait till steady conditions are reached, because even when the duration of the pumping is small, aquifer parameters can be computed by using the appropriate methods/formulas. The test results consist of recording of the drawdowns in the observation wells, which can be used to compute the aquifer characteristics by using the appropriate theoretical formulas, developed by Dupuit, Thiem, Thies, etc., as shall be discussed in the following articles.

THEORETICAL APPROACH TO COMPUTATION OF COEFFICIENT OF PERMEABILITY (K) AND DISCHARGE CAPACITY OF AN AQUIFER(Q)

Dupuit, Thiem, and Theis have developed the theoretical analysis of ground water flow, which is discussed in the following articles. The theory & formulas developed by them can be used to compute K value for a given aquifer by performing a field test of observing water levels in one or two observation well(s) at the known pumped discharge, in the main well. This K value of the given aquifer can be used to compute Q value for a well or a tubewell to be driven through that aquifer, at any given drawdown, such as 1 m, when it will indicate the specific capacity of the well. We will first of all derive the equations of Thiem, and then switch to the equations of Dupuit, although Dupuit's equations were put forth prior to the Thiem's equations.

16.10. Thiem's Equilibrium Formulas for Unconfined as well as Confined Aquifers.

16.10.1. Thiem's Formula for Unconfined Aquifer case *(for Watertable or Gravity wells).*

Let a non-artesian well be driven, and water pumped heavily so as to cause sufficient drawdown. When the water level in the well decreases, the water level in the neighbourhood will also fall down; forming what is called as an **inverted cone of depression** all around the well, as shown in Fig. 16.8. The base of this cone is a circle of radius R, known as the *circle of influence*, and the inclined side is known as the **drawdown curve.**

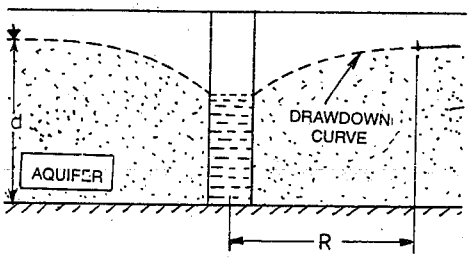


Fig. 16.8. Inverted cone of depression.

In the method suggested by Thiem, two observation wells lying within the circle of influence of the main pumped well are to be driven. Let these wells be numbered as 1 and 2 (Fig. 16.9) and let them be at distances of r_1 and r_2 from the main well (centre to centre distance). Let d be the depth of the well or the aquifer, below the static watertable.

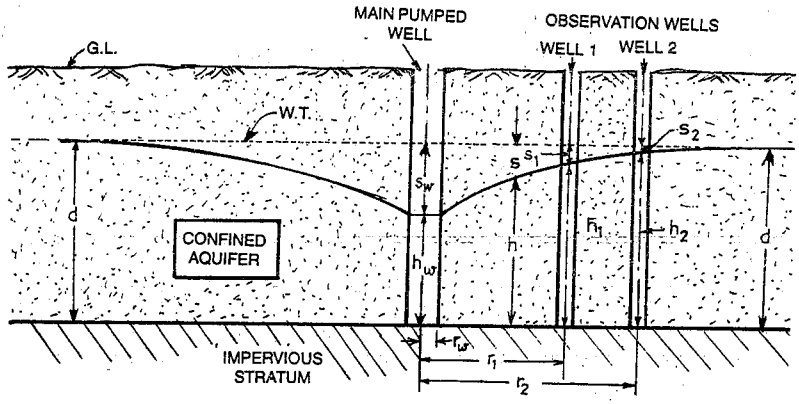


Fig. 16.9. Unconfined aquifer case of Thiem's formula.

Now, let the main well be pumped at a sufficient rate, so as to cause heavy drawdown. Then, let the pumping be so adjusted that the equilibrium conditions are reached. In other words, the rate of pumping becomes equal to the rate of yield, and thus causing the water level to attain a constant value. Since the formula involves the use of equilibrium conditions, it is known as the *equilibrium formula*. Let s_1 and s_2 be the drawdowns in the two corresponding observation wells, at this equilibrium stage.

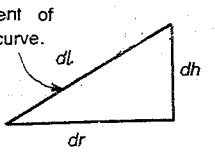
From Darcy's law, the flow through any concentric cylindrical section of the water bearing material is given by Eq. (16.2) as :

$$Q = KIA \qquad \text{where } I = \text{Hydraulic gradient}$$

Using cylindrical co-ordinates, we take r as the radius of any cylinder, and h as the height of the cone of depression at a distance r from the main well.

Assuming that the inclination of the water surface is small[*], so that the tangent can be used in place of sine for the hydraulic gradient in Darcy's law, we have

$$I = \frac{dh}{dl} \approx \frac{dh}{dr}.$$



$$\therefore \quad I = \frac{dh}{dr}$$

Also *assuming that the water flows through the full height of the aquifer (below the W.T. of course) and also that the flow is radial and horizontal (by horizontal flow we mean that the velocity distribution is assumed to be uniform as shown in Fig. 16.10).*[*], the area of flow (A) is equal to $2\pi rh$.

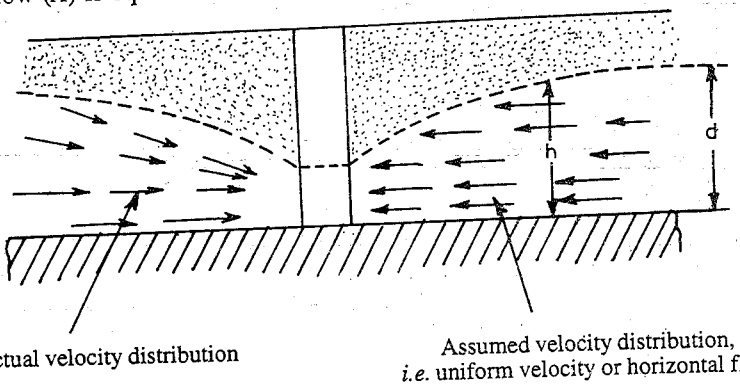


Fig. 16.10.

Substituting these values of I and A in Darcy's law, we get

$$Q = KIA = K \cdot \frac{dh}{dr} \cdot 2\pi r \cdot h$$

or $$Q = 2\pi K.h.r. \frac{dh}{dr}$$

or $$\frac{dr}{r} = \frac{2\pi Khdh}{Q}$$

Integrating between the limits r_1 and r_2 and h_1 and h_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \int_{h_1}^{h_2} \frac{2\pi K}{Q} \cdot hdh$$

where Q = Constant when steady conditions have reached.

K = Permeability of soil, which is assumed to be constant *at all places and at all times i.e.* (homogeneous soil), and in all directions, *i.e.* (isotropic soil), by assuming the soil to be homogeneous and isotropic

* Evidently, the above assumptions shall be quite true to the actual conditions in most of the flow region except in the immediate neighbourhood of the well.

Therefore $$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_1}^{h_2} h dh$$

or $$\left| \log_e r \right|_{r_1}^{r_2} = \frac{2\pi K}{Q} \left| \frac{h^2}{2} \right|_{h_1}^{h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi K}{Q} \left[\frac{(h_2^2 - h_1^2)}{2} \right] = \frac{\pi K}{Q} \left[h_2^2 - h_1^2 \right]$$

or $$K = \frac{Q \cdot \log_e \frac{r_2}{r_1}}{\pi (h_2^2 - h_1^2)}$$

or $$\boxed{Q = \frac{\pi K (h_2^2 - h_1^2)}{2.3 \log_{10} \frac{r_2}{r_1}}} \quad \text{...(16.13)}$$

This important Thiem's formula can be further simplified, if required, as follows :

$$(h_2^2 - h_1^2) = (h_2 + h_1) \cdot (h_2 - h_1)$$

But $$h_2 - h_1 = s_1 - s_2$$

and if the amount of drawdown is small compared to the saturated thickness of the water bearing material, then h_2 and h_1 are nearly equal and each is approximately equal to this saturated thickness, say d.

Therefore, $h_1 + h_2 \approx d + d = 2d$

or $$(h_2^2 - h_1^2) \approx (s_1 - s_2)\, 2d = 2d\,(s_1 - s_2)$$

Putting this value in equation (16.13), we get

$$Q \approx \frac{2\pi \cdot Kd\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad \text{...(16.14)}$$

Introducing the coefficient of transmissibility (T) instead of K, *i.e.* $T = Kd$, we have

$$Q \approx \frac{2\pi\, T\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \quad \text{...(16.14}a\text{)}$$

This is another form of Thiem's formula for the unconfined aquifers. The various assumptions that we have made in its derivation are again summarised below :

1. The aquifer is homogeneous, isotropic and of infinite and areal extent, so that its coefficient of transmissibility or permeability is constant everywhere.

2. The well has been sunk through the full depth of the aquifer and it receives water from the entire thickness of the aquifer.

3. Pumping has continued for a sufficient time at a uniform rate, so that the equilibrium stage or steady flow conditions have reached.

4. Flowlines are radial and horizontal, and flow is laminar.

5. The inclination of the water surface is small so that its tangent can be used in place of sine for the hydraulic gradient in Darcy's equation.

16.10.2. Thiem's Formula for Confined Aquifer Case. (*For Pressure or Artesian wells*) The above formula has to be slightly modified in the case of an artesian aquifer. The conditions of a confined aquifer case are shown in Fig. 16.11.

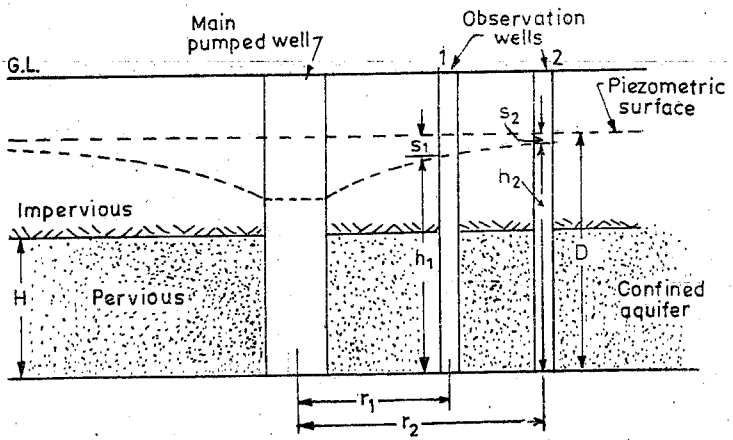


Fig. 16.11. Confined aquifer case for Theim's formula.

In a confined aquifer, the flow is actually radial and horizontal (Refer Fig. 16.12) and, therefore, it has not to be assumed as such, as it was in the unconfined case. Rest of the assumptions remain the same and hold good in this case also.

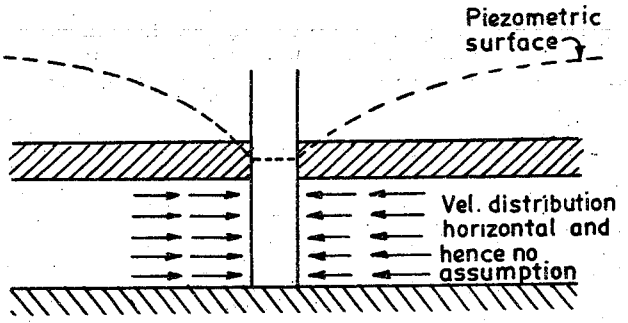


Fig. 16.12.

$$Q = KIA = K \cdot \frac{dh}{dr} 2\pi rH, \text{ because}$$

water is drawn from the cylinder of radius r and Height $= H$.

where H = height of the confined aquifer.

Now $$Q = 2\pi K.H.r. \frac{dh}{dr}$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q} \cdot dh$$

Integrating between the limits of r_1 and r_2, we get

$$\int_{r_1}^{r_2} \frac{dr}{r} = \frac{2\pi KH}{Q} \int_{h_1}^{h_2} dh$$

or $$\left| \log_e r \right|_{r=r_1}^{r=r_2} = \frac{2\pi KH}{Q} \left| h \right|_{h=h_1}^{h=h_2}$$

or $$\log_e \frac{r_2}{r_1} = \frac{2\pi KH}{Q} \left[h_2 - h_1 \right]$$

or $$\boxed{Q = \frac{2\pi KH\,(h_2 - h_1)}{2.3 \log_{10} \frac{r_2}{r_1}}} \qquad ...(16.15)$$

But $h_2 - h_1 = s_1 - s_2$

Therefore, $$Q = \frac{2\pi KH\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(16.16)$$

Also $$Q = \frac{2\pi T\,(s_1 - s_2)}{2.3 \log_{10} \frac{r_2}{r_1}} \qquad ...(16.16\,a)$$

$[\because \quad KH = T]$

where $s_1 - s_2$ is the difference of water levels between the two observation wells after the steady conditions have reached.

Limitations. Various assumptions have been made in the derivation of the above Thiem's formulae. In actual practice, however, none of these conditions may get fulfilled; say for example, an aquifer is not fully homogeneous, or the well might have been dug half way through the aquifer, or permeability may not be uniform, or the ground watertable may be inclined and thus, the base of the cone may not be a circle, or the equilibrium conditions might have not fully reached.

However, it is very difficult to assess the effects of these factors, and despite the various limiting assumptions, Thiem's formula is widely used in ground water problems and many of its limitations are removed by appropriate adjustments.

16.11. Dupuit's Original Equilibrium Formulas

The formulae put forward by Thiem and which have been derived earlier for a gravity well as well as for a pressure well are the refined forms of the original Dupuit's formulae.

We will now discuss the original Dupuit's formulas. In Dupuit' formulas, no observation wells (as constructed in Thiem's formula) are constructed. The main well is pumped out so as to get sufficient drawdown, and then the rate of pumping is so adjusted as to establish equilibrium conditions (*i.e.* the rate of inflow becomes equal to the rate of outflow, and the water level in the well becomes constant).

All the assumptions which have been made in the Theim's formulas hold good for the Dupuit's formulas also. The only difference is that the integration which was done

between the limits of r_1 and r_2 (radii of two observation wells) in Theim's formulas is changed, and the integration is done between the limits r_w and R, where r_w is the radius of the main pumped well and R is the radius of influence.

The **radius of influence** is the distance from the centre of the pumped well to the point, where the drawdown is zero or is inappreciable. The complete derivations of the Dupuit's formulae for a gravity well as well as for a pressure well are given below :

16.11.1. Dupuit's Formula for Gravity Well or Unconfined Acquifer Case (Refer Fig. 16.13).

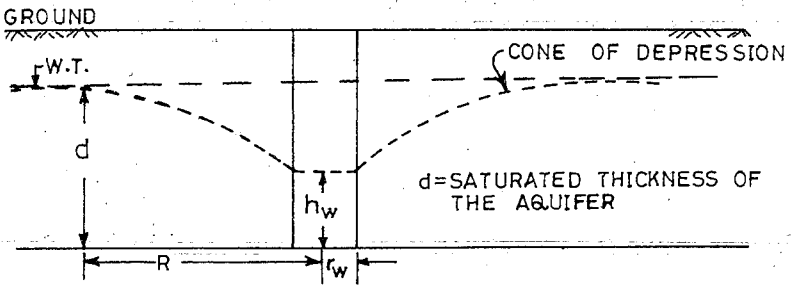


Fig. 16.13. Unconfined aquifer case for Dupuit's formula.

$$Q = K.I.A.$$

or $$Q = K \cdot \frac{dh}{dr} \cdot 2\pi rh$$

where K = Coefficient of permeability

or $$\frac{dr}{r} = \frac{2\pi K}{Q} \cdot h \cdot dh.$$

Integrating between the limits r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi K}{Q} \int_{h_w}^{d} h.dh$$

or $$\left| \log_e r \right|_{r_w}^{R} = \frac{2\pi K}{Q} \left| \frac{h^2}{2} \right|_{h_w}^{d}$$

or $$\log_e \frac{R}{r_w} = \frac{\pi K}{Q} \cdot \left[d^2 - h_w^2 \right]$$

or $$2.3 \log_{10} \frac{R}{r_w} = \frac{\pi K}{Q} \left[d^2 - h_w^2 \right]$$

or $$K = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{\pi (d^2 - h_w^2)}$$

or $$\boxed{Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10} \frac{R}{r_w}}} \quad \text{...(16.17)}$$

Since the value of R is not easily assessible, various arbitrary values have been assigned to R by various investigators. Slitcher gives it as 500 ft. (150m) and Tolman

calls it as 1000 ft. (300 m). But a more realistic picture is obtained ; when $R = CQ$, or $R \propto Q$

where, C = is a constant and
Q = is the discharge.

Putting $R = CQ$ in equation (16.17), we get

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10} \left(\frac{CQ}{r_w} \right)} \qquad ...(16.18)$$

Q can be determined by Hit and Trial method.

16.11.2. Dupuit's Formula for Pressure Well or Confined Aquifer Case. (Refer Fig. 16.14).

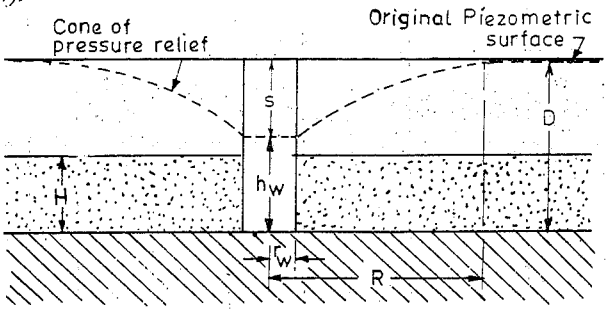


Fig. 16.14. Confined aquifer case for Dupuit's formula.

$$Q = KIA$$

or $$Q = K \cdot \frac{dh}{dr} 2\pi \cdot rH$$

or $$\frac{dr}{r} = \frac{2\pi KH}{Q} \cdot dh$$

Integrating between r_w and R, we get

$$\int_{r_w}^{R} \frac{dr}{r} = \frac{2\pi K}{Q} \cdot H \int_{h_w}^{D} dh ;$$

where D = depth of the well or height of the aquifer below the original peizometric surface

or $$\left| \log_e r \right|_{r=r_w}^{r=R} = \frac{2\pi K}{Q} \cdot H \left| h \right|_{h=h_w}^{h=D}$$

or $$2.3 \log_{10} \frac{R}{r_w} = \frac{2\pi KH}{Q} \left[(D - h_w) \right]$$

or $$K = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi . H . (D - h_w)}$$

or $$\boxed{Q = \frac{2\pi KH (D - h_w)}{2.3 \log_{10} \frac{R}{r_w}}} \qquad ...(16.19)$$

or $$Q = \frac{2\pi KHs}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} \quad ...(16.20)$$

where H = total height of the confined aquifer.

h_w = artesian pressure in the well.

r_w = radius of the well.

D = initial artesian pressure at the bottom of the aquifer or the initial height of the piezometric surface from the bottom of the well.

s = drawdown = $(D - h_w)$

16.12. Partial Penetration of an Aquifer by a Well

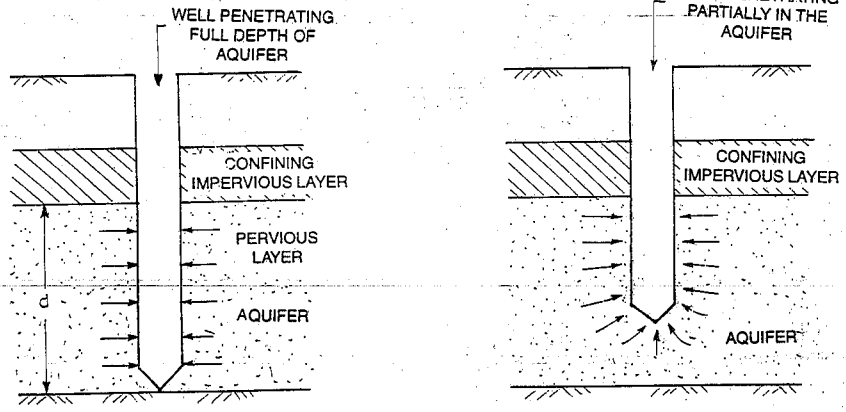
From eqn. (16.17), we have

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} \quad \text{(for unconfined wells)}$$

and from eqn. (16.19), we have

$$Q = \frac{2\pi KH (D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} \quad \text{(for confined wells)}$$

The above equations have been derived, respectively for a gravity well and for an artesian well, which penetrate throughout the aquifer. But, if a well does not penetrate up to the bottom of the aquifer, these formulas will not be applicable, as the nature of the flow will become three dimensional. It will not only be radial but will also have an upward component, as shown in Fig. 16.15. The yield of such a well is found to be more than that of a fully penetrating well of the same depth.

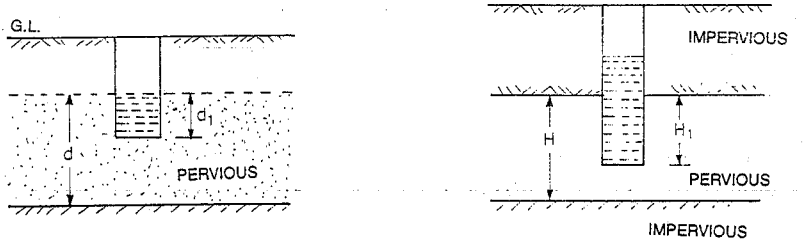


(*a*) Showing flow lines in a well fully penetrating the aquifer.

(*b*) Showing flow lines in a well partly penetrating the aquifer.

Fig. 16.15.

Kozeny has given a correction factor, and according to him the discharge Q_P through such a well is given as follows :



(*a*) Partially penetrating gravity well. (*b*) Partially penetrating artesian well.

Fig. 16.16.

Discharge (Q_P) for a partially penetrating gravity well (Fig. 16.16 (*a*)].

$$= \left[\frac{\pi \cdot K \cdot (d_1^2 - h_w^2)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right]\left[1 + 7 \cdot \sqrt{\frac{r_w}{2d_1}} \cdot \cos\frac{\pi d_1}{2d}\right] \quad ...(16.21)$$

where d_1 = Actual penetration depth below the watertable

d = Actual depth of the aquifer below the watertable.

Similarly, the discharge (Q_P) for a partially penetrating artesian well [Fig. 16.16 (*b*)].

$$= \left[\frac{2\pi \cdot KH_1 (D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right]\left[1 + 7 \cdot \sqrt{\frac{r_w}{2H_1}} \cdot \cos\frac{\pi H_1}{2H}\right] \quad ...(16.22)$$

where H = Full depth of the confined aquifer.

H_1 = Depth upto which the well penetrates.

D, h_w, r and R have the same meaning as given earlier.

16.13. Spherical Flow in a Well

Fig. 16.17 shows a special case of partially penetrating well, where the well just penetrates up to the top surface of the semi-infinite porous medium. In this case, $H_1 = 0$, and the equation (16.19) is not applicable because the flow towards the well is purely spherical. The discharge Q_s from such a well can be calculated from the equation,

$$Q_s = 2\pi K.r_w.(D - h_w) \quad ...(16.23)$$

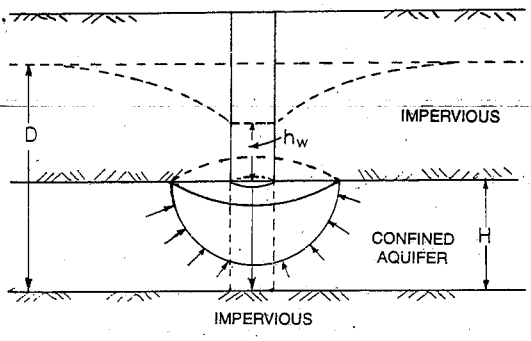


Fig. 16.17. Speherical flow in a well.

Whereas in the case of a simple radial flow in a fully penetrating

well, the discharge is given by the equation (16.19) as :

$$Q = \frac{2\pi K.H.\,(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

Now
$$\frac{Q_s}{Q} = \frac{2\pi K r_w \cdot (D - h_w)}{\left(\frac{2\pi KH\,(D - h_w)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right)} = \frac{2.3\, r_w \log_{10}\left(\frac{R}{r_w}\right)}{H}$$

or
$$\frac{Q_s}{Q} = 2.3\left(\frac{r_w}{H}\right)\log_{10}\left(\frac{R}{r_w}\right) \qquad ...(16.24)$$

For example, if

$r_w = 10\text{ cm} = 0.1\text{ m}$; and

$R = 100\text{ m}$

Then $\frac{R}{r_w} = 1{,}000$; and let

H = thickness of the confined aquifer = 20 m (say)

Then
$$\frac{Q_s}{Q} = 2.3\left(\frac{0.1}{20}\right)\log_{10}(1000)$$
$$= \frac{2.3}{200} \times 3.0 = \frac{6.9}{200} \approx \frac{1}{30}$$

This shows that the yield in a spherical flow is much less than that in a radial flow. Hence, *the spherical flow is much less efficient than the radial flow.*

16.14. Interference Among Wells

If two or more wells are constructed in such a way that they are near to each other and their cones of depressions interact, they are said to interfere. Such interference of wells decreases the discharges of such interfering wells.

Muskat has proposed the following formulas for computation of discharges from such interfering wells. These formulas have been found to yield reliable results.

(*A*) For Confined Aquifers (*i.e.* Artesian Wells)

(1) *For two artesian identical* wells at a distance B apart,*

$$Q_1 = Q_2 = \frac{2\pi KH \cdot (D - h_w)}{2.3 \log_{10}\left(\frac{R^2}{r_w \cdot B}\right)} \qquad ...(16.25)$$

(2) *For three artesian identical wells at distances B apart, in a pattern of equilateral triangle,*

$$Q_1 = Q_2 = Q_3 = \frac{2\pi KH\,(D - h_w)}{2.3 \log_{10}\left(\frac{R^3}{r_w \cdot B^2}\right)} \qquad ...(16.26)$$

* Wells of the same diameter, drawdown, and discharge over the same period of time.

(3) *For three artesian identical wells, at distances B apart in a straight line*

$$Q_1 = Q_3 = \left[\frac{\frac{2\pi KH}{2.3}(D - h_w)\log_{10}\left(\frac{B}{R_w}\right)}{\log_{10}\left(\frac{R}{B}\right)\cdot\log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right)\cdot\log_{10}\left(\frac{R}{r_w}\right)} \right] \quad ...(16.27)$$

and

$$Q_2 = \left[\frac{\frac{2\pi KH}{2.3}(D - h_w)\cdot\log_{10}\left(\frac{B}{2r_w}\right)}{2\cdot\log_{10}\left(\frac{R}{B}\right)\cdot\log_{10}\left(\frac{B}{r_w}\right) + \log_{10}\left(\frac{B}{2r_w}\right)\cdot\log_{10}\left(\frac{R}{r_w}\right)} \right] \quad ...(16.28)$$

where, Q_1 and Q_3 are the discharges of the outer-wells, and Q_2 is the discharge of the middle well.

Total discharge $= Q_1 + Q_2 + Q_3$.

(B) **For Unconfined Aquifers** (*i.e.* **Gravity Wells**)

All the above formulas can be applied to the unconfined aquifers by replacing HD by $\frac{d^2}{2}$, and $H.h_w$ by $\frac{h_w^2}{2}$.

(1) *For two identical gravity wells at distance B apart, formula would therefore, become*

$$Q_1 = Q_2 = \frac{2\pi K\left(\frac{d^2}{2} - \frac{h_w^2}{2}\right)}{2.3\log_{10}\left(\frac{R^2}{r_w \cdot B}\right)}$$

or

$$Q_1 = Q_2 = \left[\frac{\pi K(d^2 - h_w^2)}{2.3\log_{10}\left(\frac{R^2}{r_w \cdot B}\right)} \right] \quad ...(16.29)$$

(2) *For three identical gravity wells at distances B apart, in a pattern of equilateral triangle, formula would be*

$$Q_1 = Q_2 = Q_3 = \frac{2\pi K\left(\frac{d^2}{2} - \frac{h_w^2}{2}\right)}{2.3\log_{10}\left(\frac{R^3}{r_w \cdot B^2}\right)}$$

$$= \frac{\pi K(d^2 - h_w^2)}{2.3\log_{10}\left(\frac{R^3}{r_w \cdot B^2}\right)} \quad ...(16.30)$$

16.15. Surface of Seepage and Free Surface Curve

The surface throughout which the pressure is atmospheric is known as the *free surface.*

Let us consider an unconfined aquifer and let *AD* be the position of the original free water surface (*i.e.*, the watertable) which is approximately horizontal. Let a gravity well of radius r_w be constructed throughout the depth of this aquifer. Let d be the height from the bottom of the well to the ground watertable.

Let the water be pumped from this well (Fig. 16.18). After the pumping, the water will stand in the rest of the bore holes along the line *AB'C'D*, marked as "Free Surface Curve" provided the bore holes were only just deep enough to reach the free surface. Thus, in the aquifer above the surface *AB'C'D*, there is no ground water except capillary moisture.

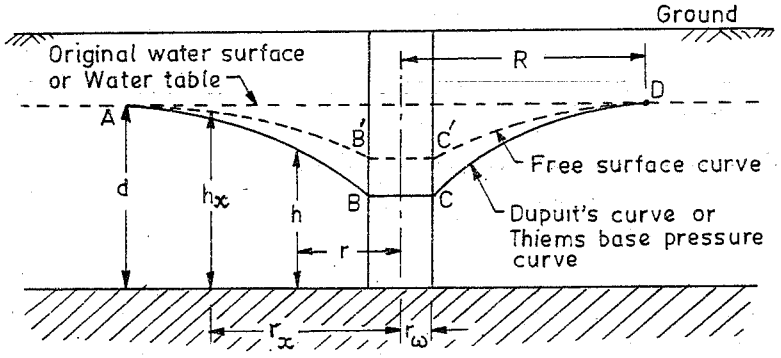


Fig. 16.18. Free surface curve Vs Dupuit's base pressure curve.

But according to the Dupuit's formula, the water level in the pumped well is not found at *B'C'* level, but at a slightly lower level, *i.e.* at *BC*, where *ABCD* is the cone of depression or Dupuit's base pressure curve. Hence, the drawdown in the pumped well is slightly more than the drawdown of the ground watertable immediately adjacent to the well [See Fig 16.19]. This vertical surface of the ground, forming outside of the well hole, which is exposed between the water surface in the well and the free surface is known as the *surface of seepage.*

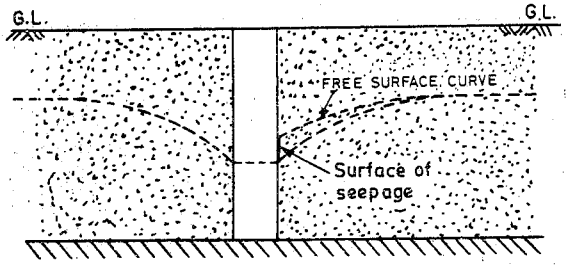


Fig. 16.19. Surface of seepage.

The difference between the actual free surface and the Dupuit's base pressure curve, in a gravity well, arises due to the Dupuit's assumption of 'horizontal and radial flow'. In other words, in an unconfined aquifer, the velocity distribution will not be horizontal near the well but Dupuit assumed it to be so, and this deviation from the actual field conditions gives rise to the difference between the actual watertable and the Dupuit's computed watertable.

The actual watertable deviates more and more from the Dupuit's computed watertable in the direction of flow, as shown in Fig. 16.20. The fact that the actual watertable

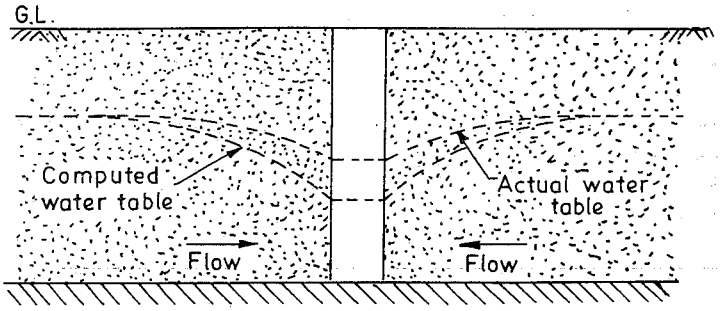


Fig. 16.20. Actual watertable and computed watertable.

lies above the Dupuit's computed watertable can be explained by the fact the Dupuit's flows are all assumed horizontal, whereas the actual prevailing velocities of the same magnitude have downward vertical component, so that a greater saturated thickness is required for the same discharge.

This discrepancy indicates that the watertable does not follow the parabolic path (as given by Dupuit). Nevertheless, for flatter slopes, where the sine and tangents nearly equal, it closely predicts the watertable position except near the outflow.

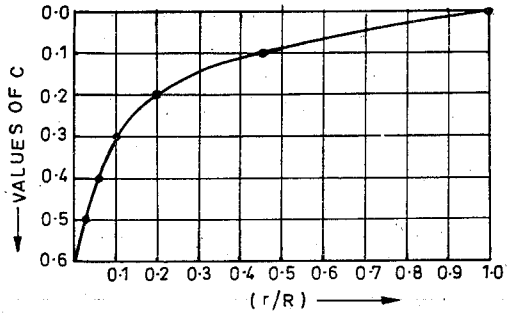


Fig. 16.21. Value of C in Eq. (16.31).

Equation of free surface curve. The free surface of water in the cone of depression surrounding a pumped well is the surface of water under the atmospheric pressure. And this free surface curve does not coincide with the Dupuit's base pressure curve but lies slightly above the same. The free surface curve is represented by the following equation:

$$Q = \left[\frac{\pi K}{2C}\right]\left[\frac{(d-h)\,d}{\log_{10}\left(\frac{R}{0.1d}\right)}\right] \qquad ...(16.31)$$

where (r, h) is any point on the curve.

R = Radius of influence.

K = Permeability coefficient.

C = a constant, the value of which depends upon the value of $\frac{r}{R}$.

The values of C in the above equation can be obtained by the curve shown in Fig. 16.21.

16.16. Well Loss and Specific Capacity

Well loss. When water is being pumped out of a well, the draw- down caused includes not only the drawdown which is given by the logarithmic drawdown curve (Equation 16.20), but also includes a certain drawdown caused by the flow of water through the well screen and axial movement within the well. The drawdown caused by

the flow through the screen and the axial movement within the well brings the water level in the well from *AB* to *CD* (Refer Fig. 16.22). This vertical drawdown *AC* or *BD* is known as the well loss. The magnitude of this well loss may be taken equal to C_2Q^2, as the flow in the vicinity of the well face is turbulent.

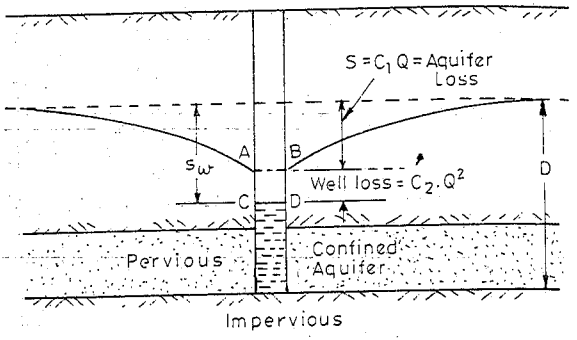


Fig. 16.22.

The total drawdown can then be obtained by adding the two drawdowns.

The first drawdown is obtained from Equation (16.20); according to which

$$Q = \frac{2\pi KHs}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

or
$$s = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi KH} = C_1 \cdot Q$$

where C_1 is a constant $= \dfrac{2.3 \log_{10} \dfrac{R}{r_w}}{2\pi KH}$

The total drawdown is then given as

$$s_w = C_1 Q + C_2 \cdot Q^2 \qquad \text{...(16.32)}$$

where C_2Q^2 = **well loss**; and

C_1Q is known as **aquifer loss** or *formation loss*

In case of an unclogged or unencrusted well screen, where the screen size is compatible to the surrounding porous media, the well loss is mainly caused by the axial movement inside the well up to the pump intake; because the well loss caused by the water entering the well through the screen is very small. However, when clogging of screen increases, the well loss increases, which adversely affects the pump efficiency.

Specific Capacity. The specific capacity of a well is defined as the well yield per unit of drawdown. Hence, the

$$\text{Sp. Capacity} = \frac{\text{discharge of the well}}{\text{Drawdown}} = \frac{Q}{C_1Q + C_2Q^2}$$

$$\therefore \quad \text{Sp. Capacity} = \left[\frac{1}{C_1 + C_2Q}\right] \qquad \text{...(16.33)}$$

The Equation (16.33) clearly shows that the sp. capacity of the well is not constant but decreases as the discharge increases.

16.17. Efficiency of a Well

It has been discussed in the previous article that discharge from a well is approximately proportional to the drawdown *s*, (where $s = C_1Q$ neglecting well loss). The

discharge per unit of drawdown was called Sp. capacity of the well; this specific capacity will be different for different well designs. For determining the best drawdown discharge conditions for a well, the well may be operated under varying drawdown conditions, and then a graph may be plotted between discharge and drawdown (called yield drawdown curve) as shown in Fig. 16.23.

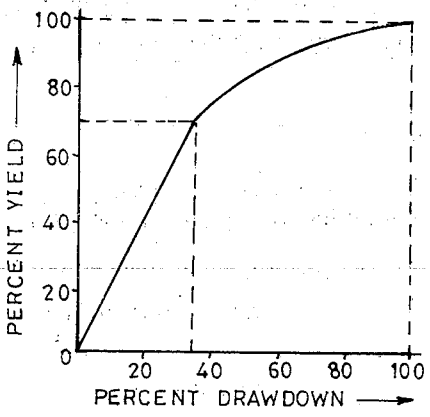


Fig. 16.23. Yield drawdown curve.

The curve obtained is a straight line up to a certain stage of drawdown, beyond which the drawdown increases disproportionately to the yield. This places an optimum and efficient limit to the drawdown, which may be allowed to be created in a well. This is generally found to be 70% of the maximum drawdown which can be created in a well.

Example 16.3. *A pumping test was made in a medium sand and gravel to a depth of 15 m where a bed of clay was encountered. The normal ground water level was at the surface. Observation holes were located at distances of 3 m and 7.5 m from the pumped well. At a discharge of 3.6 litres/sec from the pumping well, a steady state was attained in about 25 hrs. The drawdown at 3 m was 1.65 m and at 7.5 m was 0.36 m. Compute the coefficient of permeability of the soil.*

Solution. $Q = 3.6$ litres/sec

$r_1 = 3$ m; $s_1 = 1.65$ m; $h_1 = 15 - 1.65 = 13.35$ m

$r_2 = 7.5$ m, $s_2 = 0.36$ m; $h_2 = 15 = 0.36 = 14.64$ m

Using Thiem's formula for unconfined aquifers, *i.e.* equation (16.13), we get

$$Q = \left[\frac{\pi K}{2.3}\right]\left[\frac{(h_2^2 - h_1^2)}{\log_{10}\left(\frac{r_2}{r_1}\right)}\right]$$

or $$\frac{3.6}{1000}\ \text{m}^3/\text{sec} = \frac{\pi K}{2.3}\left[\frac{(14.64)^2 - (13.35)^2}{\log_{10}\left(\frac{7.5}{3}\right)}\right] = \frac{\pi K}{2.3}\left[\frac{27.99 \times 1.29}{0.398}\right]$$

or $$K = \left[\frac{2.3 \times 0.398}{\pi \times 27.99 \times 1.29}\right] \times \frac{3.6}{1000} \times 100\ \text{cm/sec} = 0.00289\ \text{cm/sec}.$$ **Ans.**

Example 16.4. *A well penetrates into an unconfined aquifer having a saturated depth of 100 metres. The discharge is 250 litres per minute at 12 metres drawdown. Assuming equilibrium flow conditions and a homogeneous aquifer, estimate the discharge at 18 metres drawdown. The distance from the well where the drawdown influences are not appreciable may be taken to be equal for both the cases.*

(U.P.S.C., Engg. Services, 1969)

Solution. $d = 100$ m

$s_1 = 12$ m $\qquad s_2 = 18$ m

$Q_1 = 250$ litres/minute, $\qquad Q_2 = ?$

Using Dupuit's formula for unconfined aquifers, *i.e.* eqn. (16.17), we have

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

In the first case ; drawdown = 12 m

$\therefore \quad h_w = (100 \text{ m} - 12 \text{ m}) = 88 \text{ m}$

$$\therefore \text{ 250 litres/minute} = \frac{\pi K [(100)^2 - (88)^2]}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}$$

(Here R and r_w are the same for both the cases)

or $$\frac{\pi K}{2.3 \log_{10}\left(\frac{R}{r_w}\right)} = \frac{250}{(100)^2 - (88)^2} = \frac{250}{188 \times 12} \quad ...(A)$$

In the 2nd case : Drawdown = 18 m

$$h_w = 100 - 18 = 82 \text{ m}$$

$$\therefore \quad Q_2 = \left[\frac{\pi K \times [(100)^2 - (82)^2]}{2.3 \log_{10}\left(\frac{R}{r_w}\right)}\right]$$

Putting the value from (A), we get

$$Q_2 = \left[\frac{250}{188 \times 12}\right][(100)^2 - (82)^2]$$

$$= \frac{250 \times 182 \times 18}{188 \times 12} = 363 \text{ litres/minute} \quad \textbf{Ans.}$$

Example 16.5. *A 30cm diameter well penetrates 25 m below the static watertable. After 24 hours of pumping @ 5400 litres/minute, the water level in a test well at 90 m is lowered by 0.53 m, and in a well 30 m away the drawdown is 1.11m. (a) What is the transmissibility of the aquifer?* *(Engg. Services, 1968)*

(b) Also determine the drawdown in the main well.

Solution. Since the well penetrates 25 m below the static watertable, it evidently is the case of an unconfined aquifer. The Thiem's discharge eqn. for such a well is given by eqn. (16.13) as:

$$Q = \frac{\pi K. (h_2^2 - h_1^2)}{2.3 \log_{10} \frac{r_2}{r_1}}$$

where $h_2 = d - s_2 = 25 - 0.53 = 24.47 \text{ m}$

$h_1 = d - s_1 = 25 - 1.11 = 23.89 \text{ m}$

$r_2 = 90 \text{ m}$

$r_1 = 30 \text{ m}$

$Q = 5400 \text{ l/min} = 5.4 \text{ m}^3/\text{min} = 0.09 \text{ m}^3/\text{s}$

Substituting the given values in the above eqn., we get

$$0.09 = \frac{3.14 \times K.\,[(24.47)^2 - (23.89)^2]}{2.3 \log_{10} \frac{90}{30}}$$

or $$K = \frac{0.09 \times 2.3 \times \log_{10} 3}{3.14\,[(24.47)^2 - (23.89)^2]} = 1.121 \times 10^{-3} \text{ m/s}$$

Now $$T = Kd = 1.121 \times 10^{-3} \times 25 \text{ m}^2/\text{s}$$

$$= 0.028 \text{ m}^2/\text{s} = \mathbf{1.68 \text{ m}^2/\text{min}} \quad \textbf{Ans.}$$

To determine the drawdown in the main well, use the above eqn. as

$$Q = \frac{\pi K (h_1^2 - h_w^2)}{2.3 \log_{10} \frac{r_1}{r_w}}$$ [using h_1 in place of h_2 and h_w in place of h_1]

or $$0.09 = \frac{1.121 \times 10^{-3}\,[(23.89)^2 - h_w^2]}{2.3 \log_{10} \frac{30}{0.15}}$$

or $$h_w = 12.08 \text{ m}$$

$\therefore$ $$s_w = d - h_w = 25 - 12.08 = \mathbf{12.92 \text{ m}}$$

Hence, the drawdown in the main well = **12.92 m** **Ans.**

Example 16.6. *60 cm diameter well is being pumped at a rate of 1360 litres/minute. Measurements in a nearby test well were made at the same time as follows. At a distance of 6 m from the well being pumped, the drawdown was 6 m, and at 15 m the drawdown was 1.5 m. The bottom of the well is 90 m below the ground watertable. (a) Find out the coefficient of permeability. (b) If all the observed points were on the Dupuit curve, what was the drawdown in the well during pumping? (c) What is the sp. capacity of the well? (d) What is the rate at which water can be drawn from this well?*

Solution. From Eqn. (16.13), we have Thiem's formula for unconfined aquifers:

$$Q = \frac{\pi K\,[h_2^2 - h_1^2]}{2.3 \log_{10} \frac{r_2}{r_1}}$$

Here $r_1 = 6$ m $\quad r_2 = 15$ m

$s_1 = 6$ m $\quad s_2 = 1.5$ m

$d = 90$ m

$Q = 1{,}360$ litres/minutes $= 1.36 \text{ m}^3/\text{min}$

$\therefore$ $h_1 = 90 - 6 \text{ m} = 84 \text{ m}$

$h_2 = 90 - 1.5 \text{ m} = 88.5 \text{ m}$

(*a*) $$1.36 = \frac{[\pi K (88.5)^2 - (84)^2]}{2.3 \log_{10} \frac{15}{6}}$$

or $$\pi K = \frac{1.36 \times 2.3 \times 0.398}{172.5 \times 4.5} = 1.603 \times 10^{-3}$$

and $$K = 0.51 \times 10^{-3} \text{ m/min} \quad \textbf{Ans.}$$

(*b*) Now $r_w = 0.3$ m

$r_2 = 15$ m

or $h_2 = 88.5$ m

$h_w = ?$

Using $$Q = \frac{\pi K (h_2^2 - h_w^2)}{2.3 \log_{10} \frac{r_2}{r_w}}$$

we get $$Q = \frac{\pi K [(88.5)^2 - h_w^2]}{2.3 \log_{10} \frac{15}{0.3}}$$

But $Q = 1.36$

and $\pi K = 1.603 \times 10^{-3}$

$\therefore$ $$1.36 = \frac{1.603 \times 10^{-3} [(88.5)^2 - h_w^2]}{2.3 \log_{10} (50)}$$

or $$\frac{1.36 \times 2.3 \times 1.69}{1.603 \times 10^{-3}} = (88.5)^2 - h_w^2$$

or $$3{,}290 = 7{,}820 - h_w^2$$

or $$h_w^2 = 7{,}820 - 3{,}290 = 4{,}530$$

or $$h_w = 67.4 \text{ m}$$

$\therefore$ Drawdown in the pumped well

$$= 90 - 67.4 = \mathbf{22.6\ m} \quad \textbf{Ans.}$$

(*c*) *Specific capacity of the well.* It is the discharge for a unit (*i.e.* 1 m) drawdown in the pumped well.

Let us first find out the value of R

Use Dupuit's equation for unconfined aquifers, *i.e.* eq. (16.17) as :

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10} \left(\frac{R}{r_w} \right)}$$

$\therefore$ $$1.36 = \frac{\pi K [(90)^2 - (67.4)^2]}{2.3 \log_{10} \left(\frac{R}{0.3} \right)}$$

or $$1.36 = \frac{1.603 \times 10^{-3} \times 157.4 \times 22.6}{2.3 \log_{10} \left(\frac{R}{0.3} \right)}$$

or $$\log_{10} \left(\frac{R}{0.3} \right) = \frac{1.603 \times 157.4 \times 22.6}{2.3 \times 1{,}360} = 1.824$$

Taking antilog, we get

$$\frac{R}{0.3} = 66.7$$

or $$R = 20.01$$

Say $$\mathbf{R = 20\ m.}$$

Now, specific capacity

$$= Q_{unitdrawdown} = \frac{\pi K [(90)^2 - (89)^2]}{2.3 \log_{10}\left(\frac{20}{0.3}\right)}$$

$$= \frac{1.603 \times 10^{-3} \times 179 \times 1}{2.3 \times 1.824} = 68.3 \times 10^{-3} \text{ m}^3/\text{min}.$$

Hence, the specific capacity

$$= \mathbf{68.3 \text{ litre/minute}} \quad \textbf{Ans.}$$

(*d*) *Maximum discharge* will occur when

$$h_w = 0$$

$$\therefore \quad Q_{max} = \frac{\pi K [(90)^2 - (0)^2]}{2.3 \log_{10}\left(\frac{20}{0.3}\right)}$$

$$= \frac{1.603 \times 10^{-3} \times 8100}{2.3 \times 1.824} = 3.09 \text{ m}^3/\text{min}$$

Hence, the maximum rate of discharge

$$= \mathbf{3{,}090 \text{ litres/minute}} \quad \textbf{Ans.}$$

16.18. Non-Equilibrium Formula for Aquifers (Unsteady Radial Flows)

The main drawback of the equilibrium formulas given by Thiem and Dupuit, was the problem to attain equilibrium conditions, since it is not an easy job to do so. The pumping has to be continued at a uniform rate for a very long time so as to achieve steady flow conditions. Further research was, therefore, carried out to simplify the process and to calculate the yield in some other way.

A major advancement in this field was made by Thies when he developed his non-equilibrium formula by introducing the time factor t. This formula was derived by Thies in 1935, by comparing the flow of water with the flow of heat by conduction. The formula evolved by him involved the solution of a complicated integral, the solution of which requires the use of various tables and graphs.

Later, Jacob derived the same formula by directly using the hydraulic concept. He also slightly modified the Thies formula by making a slight approximation, so as to simplify it. The final formula which was arrived at the end, by Jacob, is given as :

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right] \qquad ...(16.34)$$

where s = Drawdown in the observation well after a time t, from the start of pumping in the main well

T = Coefficient of transmissibility of the aquifer

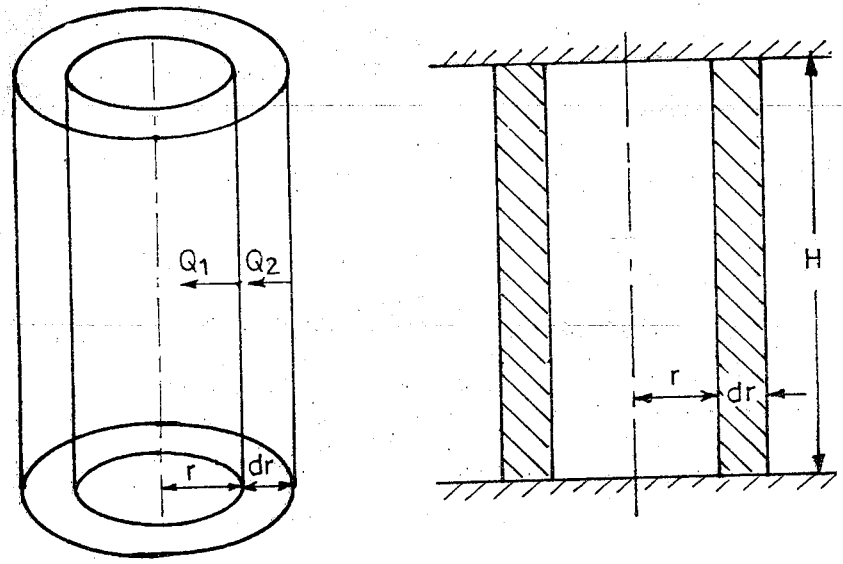
Q = Constant discharge pumped out from the main pumped well.

A = Coefficient of storage of the measured drawdown.

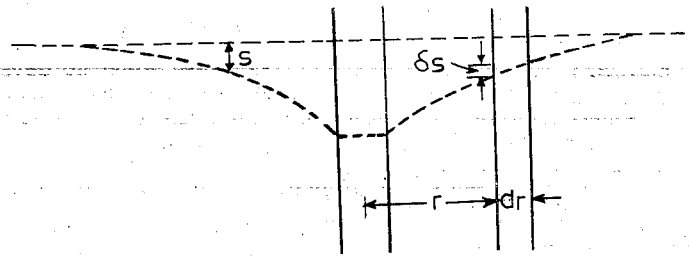
r = Radial distance of the observation well from the main pumped well.

The derivation of this formula is given below :

Derivation. Let us consider a generalised free-body diagram of the flow system in the vicinity of a discharging well, as shown in Fig. 16.24.



(*a*)



(*b*)

Fig. 16.24.

Let the impermeable planes bound the system on top and bottom, and we also assume that the flow is radial and horizontal but the equilibrium conditions are not necessary as the flow is considered to be unsteady.

Hydraulic gradient on the inner face of the cylindrical shell

$$= I_1 = \frac{\partial s}{\partial r}$$

It should be negative because as s is increasing, r is decreasing

$$\therefore \qquad I_1 = -\frac{\partial s}{\partial r}$$

Hydraulic gradient at the outer face $= I_2$

$$= \left(I_1 + \frac{\partial I_1}{\partial r} \cdot dr \right) \qquad \text{(with –ve sign)}$$

or $$I_2 = -\left(\frac{\partial s}{\partial r} + \frac{\partial^2 s}{\partial r^2} \cdot dr \right)$$

Now Q_1 = Discharge from the inner face

$$= K I_1 A$$

$$= K \left(-\frac{\partial s}{\partial r} \right) \cdot 2\pi r \cdot H \qquad \left[\because \begin{array}{l} 2\pi r = \text{Surface area} \\ H = \text{Full depth of the aquifer} \end{array} \right]$$

$$\therefore \quad Q_1 = -2\pi KH.r. \frac{\partial s}{\partial r} = -2\pi T.r. \frac{\partial s}{\partial r} \qquad [\because \; Kd = T]$$

or $$Q_1 = -2\pi rT \frac{\partial s}{\partial r} \qquad \ldots(i)$$

Similarly, $Q_2 = K I_2 A = K I_2 2\pi (r + dr) H = T . I_2 2\pi (r + dr)$

$$= -T \left(\frac{\partial s}{\partial r} + \frac{\partial^2 s}{\partial r^2} dr \right) 2\pi (r + dr) \qquad \ldots(ii)$$

subtracting (*ii*) from (*i*), we get

$$Q_1 - Q_2 = -2\pi rT \cdot \frac{\partial s}{\partial r} + T . \left[\frac{\partial s}{\partial r} + \frac{\partial^2 s}{\partial r^2} \cdot dr \right] 2\pi (r + dr)$$

$$= -2\pi rT. \frac{\partial s}{\partial r} + 2\pi rT \frac{\partial s}{\partial r} + 2\pi r.T \frac{\partial^2 s}{\partial r^2} dr. + 2\pi.T.dr \frac{\partial s}{\partial r} + 2\pi T.d_r^2 \frac{\partial^2 s}{\partial r^2}$$

$$= 2\pi rT. \frac{\partial^2 s}{\partial r^2} dr + 2\pi T dr \cdot \frac{\partial s}{\partial r} + 2\pi T.d_r^2 \cdot \frac{\partial^2 s}{\partial r^2}$$

Neglecting $2\pi.T.\frac{\partial^2 s}{\partial r^2} d_r^2$, as the differentials of the higher order are very small, we get

$$Q_1 - Q_2 = 2\pi rT. \frac{\partial^2 s}{\partial r^2} \cdot dr + 2\pi T.dr. \frac{\partial s}{\partial_r} = 2\pi T. \left[r . \frac{\partial^2 s}{\partial r^2} dr + \frac{\partial s}{\partial r} .dr \right]$$

$$= 2\pi rT.dr. \left[\frac{\partial^2 s}{\partial r^2} + \frac{1}{r} \frac{\partial s}{\partial r} \right] \qquad \ldots(iii)$$

$Q_1 - Q_2$ (*i.e.* the difference in the rate of flow through the inner and outer faces of the cylindrical shell) must be drawn from the storage within the shell (Principle of Conservation of Mass).

Therefore, $Q_1 - Q_2$ = Rate of change of volume of this storage of the thin cylindrical shell of thickness *dr*.

$$= \frac{(2\pi r.dr) .A.\partial s}{\partial t}$$

[Because A = Coeff. of storage, *i.e.* the volume of water drained through a unit-cross-section and full depth, when pressure falls by unity.

In this case,

pressure fall is equal to ∂s; and the total volume drained = (surface area drained) $\times A \times$ fall of pressure

or Change of volume in a time ∂t $= (2\pi r.dr.)\, A.\partial s$

or Rate of change of volume $= \dfrac{2\pi r.dr.A.\partial s}{\partial t}$]

Equating these two values of $Q_1 - Q_2$ given by eqn. (*iii*) and (*iv*), we get

$$2\pi rT.dr.\left[\frac{\partial_s^2}{\partial_r^2}+\frac{1}{r}\cdot\frac{\partial s}{\partial r}\right]=\left[2\pi r.dr.A.\frac{\partial s}{\partial r}\right]$$

or

$$\frac{\partial_s{}^2}{\partial_r{}^2}+\frac{1}{r}\cdot\frac{\partial s}{\partial r}=\frac{A}{T}\cdot\frac{\partial s}{\partial t}$$

This is the differential equation governing this unsteady flow and can be solved for the boundary conditions of our given problem *i.e.* $s = 0$ before pumping begins and s approaches zero as r approaches infinity after pumping begins, *i.e.* $s = 0$ at $t = 0$ and $s \to 0$ as $r \to \infty$.

The solution of this differential equation for a constant pumping rate Q is given by the expression.

$$s=\frac{Q}{4\pi T}\int_{x=\frac{r^2A}{4Tt}}^{x=\infty}\frac{e^{-x}}{x}\cdot dx$$

The above solution is obtained after complicated mathematical calculations, which are beyond the scope of this book. The integral involved above is called **well-function.**

Now we can again write

$$s=\frac{Q}{4\pi T}\int_{x=\frac{r^2A}{4Tt}}^{x=\infty}\frac{e^{-x}}{x}\cdot dx=\frac{Q}{4\pi T}\left[\int\frac{e^{-x}}{x}\cdot dx\right]_{\text{at } x=\frac{r^2A}{4Tt}} \quad \because \quad \left|\int\frac{e^{-x}}{x}dx\right|_{\text{at } x=\infty}=0$$

But $\int\frac{e^{-x}}{x}dx$ is given by the following series:

$$\int\frac{e^{-x}}{x}dx=\left[-0.5772-\log_e x+x-\frac{x^2}{2\lfloor 2}+\frac{x^3}{3\lfloor 3}\cdots\right]$$

Hence, the drawdown,s, after a given time t and at a distance r from the centre of the pumped well is given by the equation

$$s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e x + x - \frac{x^2}{2\lfloor 2} + \frac{x^3}{3\lfloor 3} + \ldots\right] \quad \ldots(16.34a)$$

where $x = \dfrac{r^2 \cdot A}{4T \cdot t}$

This equation, as derived by Thies, involved complicated mathematical calculations and was later simplified by Jacob as follows :

Modified Non-Equilibrium Formula. It is evident that x decreases as t increases, and Jacob found that the terms beyond $\log_e x$ in equation (16.34*a*) are not appreciable and can be neglected for larger values of t. Using this approximation, equation (16.34*a*) can be reduced to

$$s = \frac{Q}{4\pi T}\left[-0.5772 - \log_e x\right] \text{ where } x = \frac{r^2 A}{4Tt}$$

$$= \frac{Q}{4\pi T}\left[\log_e \frac{1}{x} - 0.5772\right] \text{ where } x = \frac{r^2 A}{4Tt}$$

or
$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 A} - 0.5772\right] \quad \ldots(16.34)$$

This is the important final equation and must be remembered. If, in an observation well at a distance r from the main pumped well, the drawdowns are respectively s_1 and s_2 at times t_1 and t_2 after the pumping was started in the main well, then

$$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r^2 . A} - 0.5772\right] \quad \ldots(v)$$

and
$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - 0.5772\right] \quad \ldots(vi)$$

Subtracting (v) from (vi), we get

$$s_2 - s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} - \log_e \frac{4Tt_1}{r^2 A}\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r^2 A} \cdot \frac{r^2 A}{4Tt_1}\right]$$

$$= \frac{Q}{4\pi T}\log_e \frac{t_2}{t_1} = \frac{2.3Q}{4\pi T}\log_{10} \frac{t_2}{t_1}$$

or
$$s_2 - s_1 = \frac{2.3Q}{4\pi T}\log_{10} \frac{t_2}{t_1} \quad \ldots(16.35)$$

The above formula holds good for larger values of t. It is evident from the above equation that if the drawdowns are noted for various values of t on the given observation well at a distance r from the main pumped well, and a graph is plotted between log t and s, it will be a straight line, with the limitation that the initial values (*i.e.* when t is small) may not exactly lie on a straight line. From this straight line, two values, of t_1 and t_2 and corresponding values of s_1 and s_2 can be read out, and knowing the value of T, Q can be worked out easily.

The straight line can be produced so as to cut the X-axis at point P as shown in Fig. 16.25. Now let the value of t at P be represented by t_0.

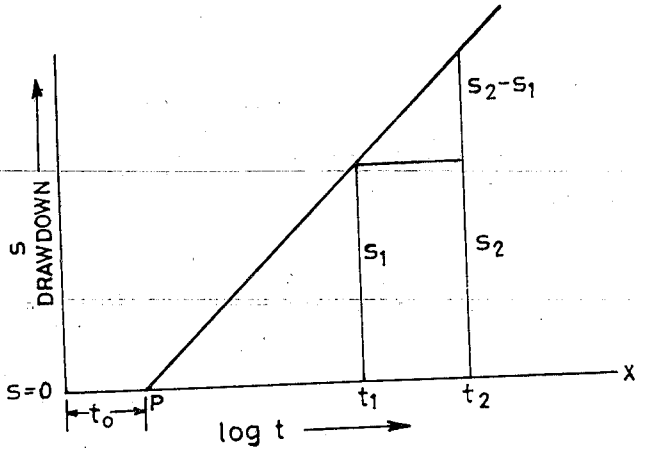


Fig. 16.25.

Now, we know that

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right]$$

at $s = 0, t = t_0$

$$\therefore \quad 0 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_0}{r^2 A} - 0.5772\right]$$

Since $\frac{Q}{4\pi T} \neq 0$

$$\log_e \frac{4Tt_0}{r^2 A} - 0.5772 = 0$$

or $$\log_e \frac{4Tt_0}{r^2 A} = 0.5772$$

or $$\frac{4Tt_0}{r^2 A} = e^{0.5772}$$

or $$A = \frac{4Tt_0}{r^2 e^{0.5772}} = \frac{2.25Tt_0}{r^2}$$

Hence, $$\boxed{A = \frac{2.25Tt_0}{r^2}} \qquad ...(16.36)$$

The coefficient of storage A, can be worked out by this equation.

Example 16.7. *In an artesian aquifer, the drawdown is 1.2 metres at a radial distance of 10 metres from a well after two hours of pumping. On the basis of Thies' non-equilibrium equation, determine the pumping time for the same drawdown (i.e. 1.2 m) at a radial distance of 30 metres from the well.*

Solution. The Thies' non-equilibrium equation (16.34) is

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 . A} - 0.5772\right]$$

In the given question, the drawdown is the same in both the observation wells, therefore,

well (1) $r_1 = 10$ m, $t_1 = 2$ hr. well (2) $r_2 = 30$ m, $t_2 = ?$

Now $$s_1 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 . A} - 0.5772\right]$$

$$s_2 = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

But $s_1 = s_2$

$$\therefore \quad \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_1}{r_1^2 A} - 0.5772\right] = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt_2}{r_2^2 A} - 0.5772\right]$$

or $$\log_e \frac{4Tt_1}{r_1^2 A} = \log_e \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{4Tt_1}{r_1^2 A} = \frac{4Tt_2}{r_2^2 A}$$

or $$\frac{t_1}{r_1^2} = \frac{t_2}{r_2^2}$$

Putting the respective values, we get

or $$\frac{2 \text{ hr}}{(10)^2} = \frac{t_2}{(30)^2}$$

or $$t_2 = \frac{(30)^2}{(10)^2} \times 2 \text{ hr} = 9 \times 2 \text{ hr} = 18 \text{ hr}$$

$$\mathbf{t_2 = 18 \text{ hr}} \quad \textbf{Ans.}$$

Example 16.8. *A well is located in a 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minute, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping.* *(U.P.S.C. Civil Services, 1991)*

Solution. Using Jacob's eqn. (16.34), we have

$$s = \frac{Q}{4\pi T}\left[\log_e \frac{4Tt}{r^2 A} - 0.5772\right]$$

where s = drawdown = ?

H = depth of aquifer = 30 m

$K = 35$ m/day

A = storage coeff = 0.004

$Q = 1500\ l/\text{min} = \frac{1.5}{60}\ \text{m}^3/\text{sec} = \mathbf{0.025\ m^3/s}$

$r = 40$ m

$t = 20 \text{ hr} = 20 \times 3600 \text{ secs} = 72000 \text{ secs}$

$$T = K \,.\, H = 35 \times 30 \text{ m}^2/\text{day}$$
$$= \frac{1050}{60 \times 60 \times 24} \text{ m}^2/\text{sec} = 0.012153 \text{ m}^2/\text{s}$$

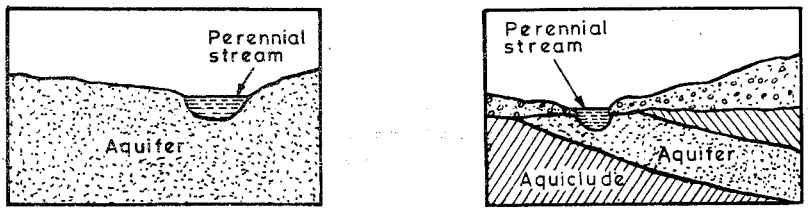
Substituting values, we get

$$s = \frac{0.025}{4 \times 3.14 \times 0.012153}\left[\log_e \frac{4 \times 0.012153 \times 72000}{(40)^2 \times 0.004} - 0.5772\right]$$
$$= 0.163\,[6.3042 - 0.5772] = 0.163 \times 5.724 = \mathbf{0.94\ m.} \quad \textbf{Ans.}$$

16.19. The Method of Images - Its Use in Ground Water Analysis for Areally Limited Aquifers.

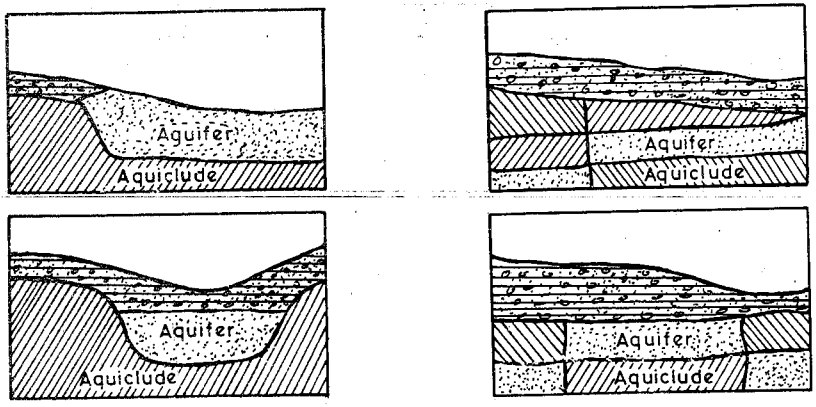
The equilibrium as well as the non-equilibrium formulas, derived in the previous articles, for analysing ground water pumping, have been derived on the assumption that the aquifer being pumped is of *infinite areal* extent. This assumption is generally not fulfilled in the actual field except in the cases of a few sedimentary rock aquifers. *In actual practice, the continuity of aquifers is usually broken by the existence of formation boundaries, folds & faults, or the dissection by surface streams.* The existence of such geological structures limit the areal extent of aquifers to a few kilometers or more in consolidated strata. In unconsolidated materials and particularly in the glaciated areas, the infinite areal extent is seldom met with. Consequently, it becomes necessary to consider the effects caused by the existence of such geological structures on the movement of ground water, before the foregoing formulas (derived in the previous articles) can be applied to the problems of flow in *really limited aquifers.*

Thus, when an aquifer is intersected by a perennial stream or some other water body with sufficient flow, it will prevent development of the cone of depression beyond that surface



(*i*) Unconsolidated strata (*ii*) Consolidated strata

(*a*) Positive boundary caused by the intersection of an aquifer by a perennial stream.



(*i*) Unconsolidated strata (*ii*) Consolidated strata

(*b*) Negative boundary caused by the intersection of an aquifer by impervious strata.

Fig. 16.26. Positive and Negative geological boundaries in aerially llmited aquifers.

source. The intersection of the aquifer with the stream will then become a geological boundary, and is called a **positive boundary**, since it will help in preventing drawdowns over large areas. On the other hand, when some impervious aquiclude formation does intersect an aquifer, it will stop the flow from the aquifer area bounded by it, and hence such a contact is known as a **negative boundary**. Examples of positive & negative contacts are reflected in Fig. 16.26 (*a*) and (*b*), respectively.

Strictly speaking, most geologic boundaries except for some faulted structures, do not occur as abrupt straight line demarcations, yet they can be assumed to be so without involving much error, since the area covered by a well being pumped, is relatively small.

By considering the geologic boundary as a straight line demarcation, it has further been possible to solve the flow pattern by substitution of a hypothetical system that satisfies the limits of the real system. The **method of images**, devised byLlord Kelvin in his work on electrostatic theory, is a convenient tool for the solution of boundary problems.

Let us for example, consider an idealized section of an aquifer which is inter-sected by a surface stream in Fig. 16.27. To act as an effective positive boundary, the flow in the surface stream must be equal or exceed the withdrawal of the well, because any flow

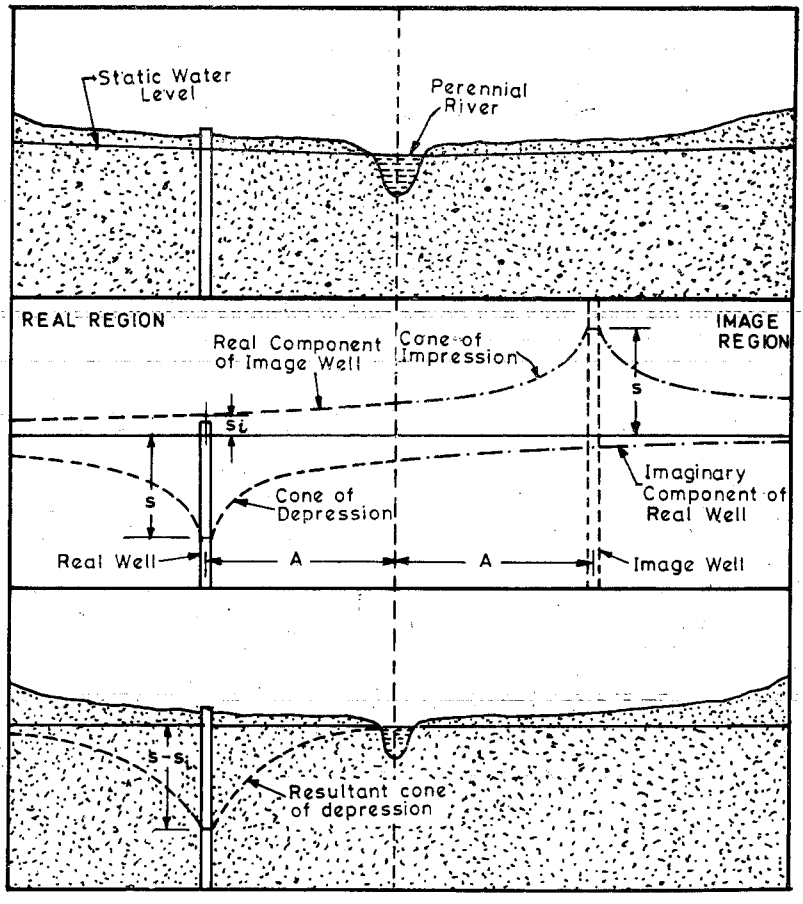


Fig. 16.27. Idealized section of an aquifer which is intersected by a perennial stream alongwith a hypothetical well system for the solution of this type of flow problems.

below the well yield would result in drying up of the stream, eliminating the said boundary. The stream is now assumed to be of infinitesimal width, *i.e.* equivalent to a line source. At this line source, there will be zero drawdown, since the river is perennial. Any system that can satisfy this boundary limit will offer the solution to the real problem.

As shown by the central Fig. in Fig block 16.27, the real and bounded aquifer has been replaced by an *imaginary aquifer of infinite areal extent and an imaginary recharging well, called* **image well**, placed on the opposite side of and equidistant from the boundary. As illustrated, the imaginary recharge well returns water to the aquifer at the same rate as it is withdrawn by the real discharge well. The image well, consequently produces a build up of water level at the boundary that is exactly equal to, and cancels the drawdown of the real well. This system results in obtaining zero drawdown at the boundary, which satisfies the limit of the real problem.

The real components of the cone of depression of the real well and the cone of impression of the image well are shown as solid lines in the region of real values. to obtain the resultant cone of depression; *i.e.* to evaluate the drawdown at any point in the real region, we shall have to add algebraically the real components of the two depression cones. *The resultant cone of depression is thus found to be steepened on the riverside of the well and flattened on the landward side.*

Note. The actual observed drawdown in a well bounded by such a river boundary on pumping, will hence be found to be less than what is computed by the Dupuit's or Thiem's formulas i.e. s. The reduction being equal to the drawdown caused by the image well at the location of the real well at distance 2A apart ; say s_i. Hence, actual drawdown in the real well will be equal to $s - s_i$ inplace of s. It use has been exhibited in solving numerical example 16.9.

Similarly, the utilisation of an image well in solving the problem of an aquifer bounded by an impervious strata is reflected in Fig. 16.28.

Example 16.9. *A 0.5m diameter well fully penetrates a 30m thick confined aquifer of hydraulic conductivity 20m/day. Due to continuous pumping, if a drawdown of 1.0 m is registered in the well, what will be the rate of pumping when the well is located at a distance of 50 m from a perennial stream.* (*Civil Services, 1999*)

Solution. In this case, the aquifer is not of infinite areal extent, but is inter-sected by a perennial stream at a distance of 50 m from the well, which will reduce the drawdown in the well than what is computed by Dupuit's or Thiem's formulas.

If s represents the drawdown computed by Dupuit's or Thiem's formulas, and s_i represents the drawdown caused at the location of the real well by the image well (at a distance of $2A = 2 \times 50$ m = 100 m), then we can write;

The actual drawdown in the well of aerially limited aquifer

$$= s - s_i = 1.0 \text{ m (given)} \qquad ...(i)$$

Thiem's formula is now applied to the image well being pumped at a discharge of Q, to find out its impact (i.e. drawdown, s_i) at a distance $2A$, by using Eq. 16.14, as :

$$Q \approx \frac{2\pi.K.d.\,(s_1 - s_2)}{2.3 \log (r_2/r_1)} \qquad (i.e.\ \text{Eq. } 16.14)$$

where, $s_1 = s$ (image well)

$s_2 = s_i$ (impact of image well at distance 2A apart)

$r_1 = r = 0.25$ m (given), since the real well as well as its image well are of 0.5 m dia.

$r_2 = 100$ m (*i.e.* $2A$).

$d = 30$ m (given) ; and $K = 20$ m/day (given)

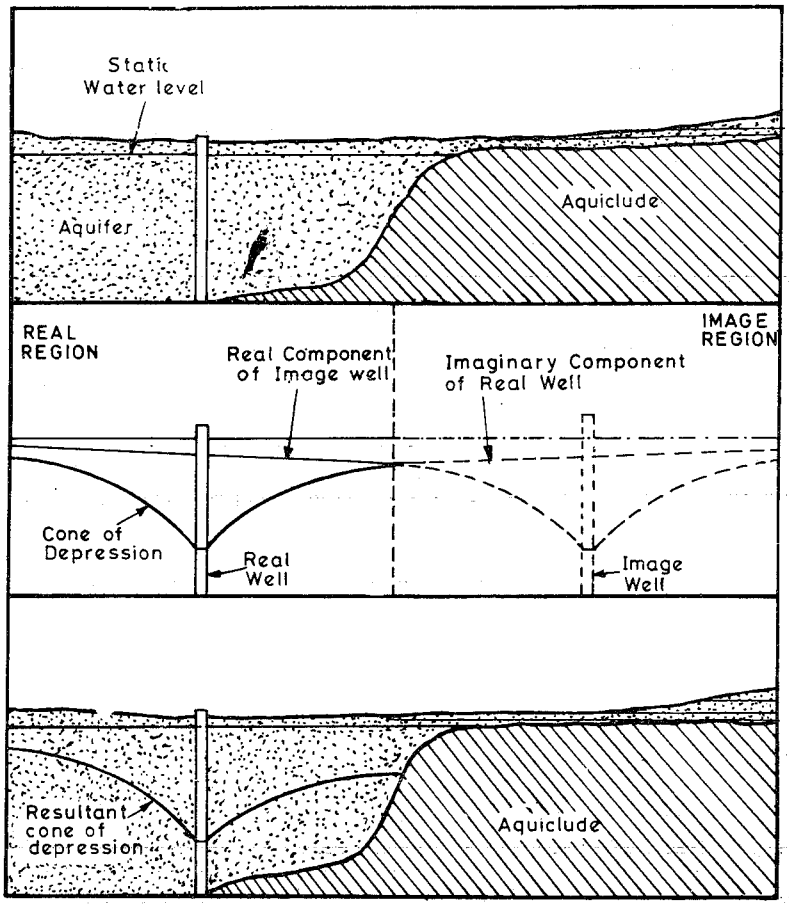


Fig. 16.28. Idealized section of an aquifer bounded by an 'impervious formation', shown along with a hypothetical well system for the solution of this type of flow problems.

Substituting the values, we get

$$Q = \frac{2\pi \times 20 \times 30\,(s - s_i)}{2.3 \log_{10} \frac{100}{0.25}}\ \text{m}^3/\text{day}$$

or $$Q = 629.92\,(s - s_i) \qquad ...(ii)$$

But from (i)

$$s - s_i = 1\ \text{m}$$

$$\therefore \qquad \mathbf{Q = 629.92\ m^3/day = 6.30\ lps.} \qquad \textbf{Ans.}$$

16.20. Recharging of Underground Storage

Just as the artificial surface reservoirs are constructed by building dams in order to store the surplus surface waters; in the same manner, artificial underground reservoirs are now-a-days developed by *artificial recharge*, for storing water underground. The development of such a reservoir may be advantageous as compared to the development of a dam reservoir, because of the following reasons;

(*i*) Much purer water can be obtained from an underground reservoir source.

(*ii*) No space is required for building such a reservoir.

(*iii*) The cost of building such a reservoir by recharging the aquifers may be considerably less than the cost of the surface reservoirs. Moreover, in an underground reservoir, the aquifer in which the water is stored shall itself act as a distribution system for carrying the water from one place to another, and as such, the necessity of constructing pipe lines or canals (as is required in a surface reservoir) is completely eliminated. On the other hand, extra cost is involved for pumping out the ground water. However, the reduction in the first cost due to elimination of a huge distribution network may sometimes be so high as to offset the subsequent cost of pumping, which is required in such underground reservoirs.

(*iv*) The water lost in evaporation from an underground reservoir is much less than the water lost from a surface reservoir.

(*v*) The raising of the watertable by artificial recharge may help in building pressure barriers to prevent sea water intrusion in the coastal areas.

Artificial recharge of ground water is, therefore, preferred and encouraged in modern days, so as to augment the natural available underground yield, for management of water supply systems. Artificial recharging technique is under intensive research, and is being increasingly used in France, Israel, Federal Republic of Germany, U.K. etc.

16.20.1. Methods of Recharging. The three methods which are generally adopted for ground water recharging are discussed below :

(1) **Spreading method.** This method consists in *spreading* the water over the surfaces of permeable open land and pits, from where it directly infiltrates to rather shallow aquifers. In this method, the water is temporarily stored in shallow ditches, or is spread over an open area by constructing low earth dykes (called *percolation bunds*). The stored water, slowly and steadily, percolates downward, so as to join the nearby aquifers. The recharging rate depends upon the permeability of the spread area and on the depth of water stored, and is generally less, say of the order of 1.5 m/day, though rates as high as 22.5 m/day have been possible. Certain chemicals, when added to the soil, may help in increasing the recharging rate and are under research.

(2) **Recharge-well method.** This method consists in injecting the water into bore holes, called recharge wells. In this method of recharge, the water is therefore, fed into recharge wells by gravity or may be pumped under pressure to increase the recharge rate, if surface conditions permit. The recharge wells used for this purpose, are just like ordinary production wells. Infact the ordinary production wells are many a times directly used for recharge during the off season, when the water is not required for use. *Recharge-well method* is certainly preferred when the *spreading method* cannot yield appreciable recharge, because of low permeable areas. High recharge rates can be obtained with this method. Moreover, this method may help in injecting water into the aquifers, and also where it is most needed. This method is widely practised in Israel.

The water to be used in the recharge well should, however, be purer than that is required in the first method. This water should be free of suspended matter, so as to avoid clogging of the well screens. Since the recharge well injects the water directly into the aquifer, the water used for recharging must also be free from bacteria. Hence, if the treated sewage is used for recharge, it should generally be bacteriologically pure.

(3) **Induced Infiltration method.** The third method which is sometimes used for recharge is that of the induced infiltration, which is accomplished by increasing the watertable gradient from a source of recharge. In this method, Renney type* wells are

* Ranney well is explained in the next article.

constructed near the river banks. The percolating water is collected in the well through radial collectors and is then discharged as recharge into a lower level aquifer 'B' for storage, as shown in Fig. 16.29.

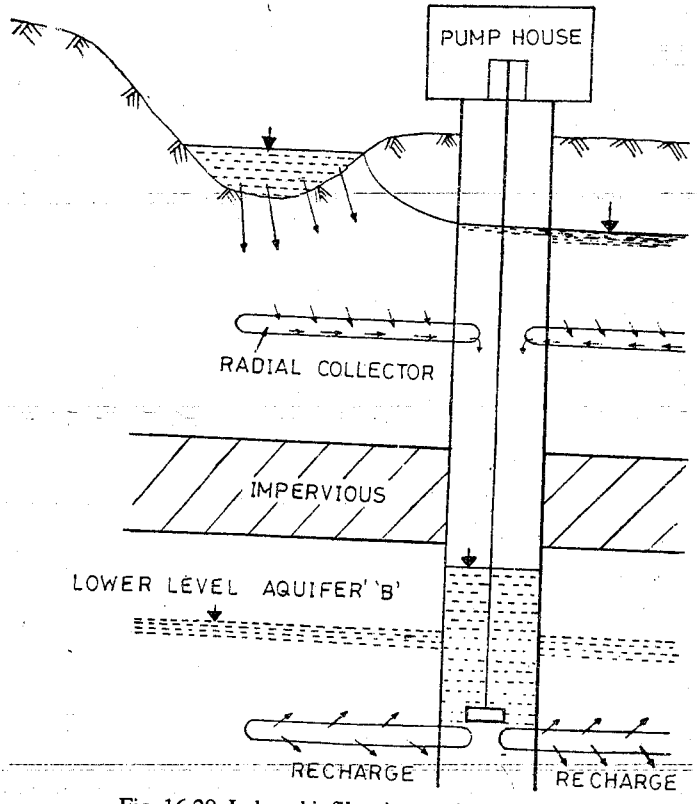


Fig. 16.29. Induced infiltration method of recharge.

VARIOUS FORMS OF UNDERGROUND SOURCES AND THEIR EXPLOITATION

The underground water is generally available in the following forms:

(1) Infiltration galleries; (2) Infiltration wells;
(3) Springs; and (4) Wells including tubewells.

These forms are discussed below :

16.21. Infiltration Galleries

Infiltration galleries are horizontal or nearly horizontal tunnels constructed at shallow depths (3 to 5 metres) along the bank of the river through the water bearing strata, as shown in Fig. 16.30(*a*). They are sometimes called *horizontal wells.*

These galleries are generally constructed of masonry walls with roof slabs, and derive their water from the aquifer by various porous drain pipes. These pipes are generally covered with gravel, so as to prevent the entry of the fine sand particles into the pipe. These tunnels or galleries are generally laid at a slope, and the water collected in them is taken to a sump well, from where it is pumped, treated and distributed to the consumers. These infiltration galleries are quite useful when water is available in sufficient quantity just below the ground level or so.

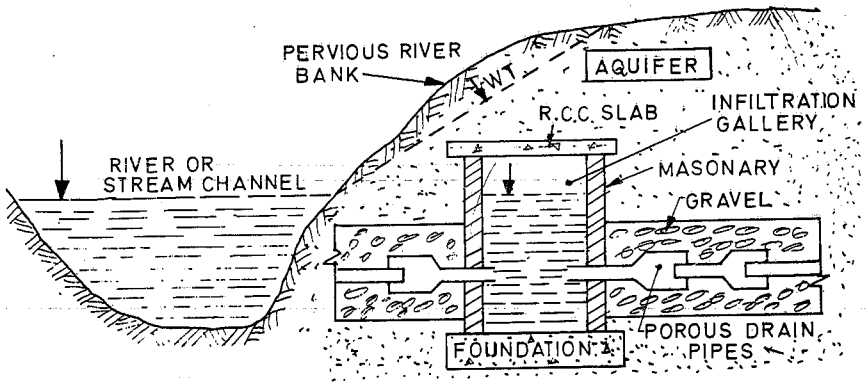


Fig. 16.30 (*a*). Section of an infiltration gallery.

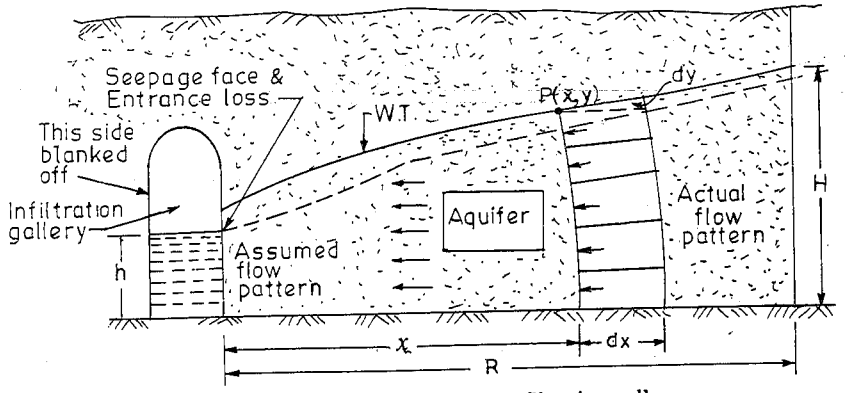


Fig. 16.30 (*b*). Flow pattern in an infiltrating gallery.

The flow pattern into an infiltration gallery under pumping equilibrium conditions is shown in Fig. 16.30 (*b*), as the drawdown curve follows the Dupuit's curve. The discharge, from the gallery can be computed by using Darcy's law, as

$$Q = K\,i\,A$$

Now, consider a point P on the drawdown curve having coordinates (x, y). On considering a small element of width dx, the head gradient will be dy/dx, and the area of soil through which flow will occur will be $y \,.\, L$, where L is the length of the gallery ($\perp$ to paper).

$$\therefore \text{ Discharge } Q = \int_{x=0}^{x=R} K \,.\, \left(\frac{dy}{dx}\right) \,.\, (y.L).$$

or
$$Q \times \int_{x=0}^{x=R} dx = K.L. \int_{y=h}^{y=H} y\,dy$$

where x varies between O to R, where R is the radius of influence.

y varies between h and H ; where

h = depth of water in the gallery on pumping at equilibrium

H = Static head *i.e.* height of initial water level above the bottom of gallery.

$$\therefore \qquad Q \left| x \right|_{x=0}^{x=R} = K.L \left| \frac{y^2}{2} \right|_{y=h}^{y=H}$$

$$\therefore \qquad Q.R = K.L \left(\frac{H^2 - h^2}{2} \right)$$

or
$$Q = K.L. \left(\frac{H^2 - h^2}{2R} \right) \qquad ...(16.37)$$

where Q = Discharge
K = Permeability coefficient of the aquifer
L = length of the gallery ($\perp$ to Fig.)
H = Initial depth of water level
h = Final depth of water level
R = Radius of influence

Sometimes, horizontal perforated pipes are laid in place of rectangular tunnels, and their perforations are covered with gravel so as to prevent sand entry. These pipes may be called infiltration pipes, and are useful when the available ground water is small in quantity.

16.22. Infiltration Wells

Infiltration wells are the shallow wells constructed in series along the banks of the rivers, in order to collect the river water seeping through their bottoms, as shown in Fig. 16.31.

These wells are generally constructed of brick masonry with open joints. They are generally covered at the top and kept open at the bottom, as shown in Fig 16.32. For inspection purposes, manholes are provided in the top cover.

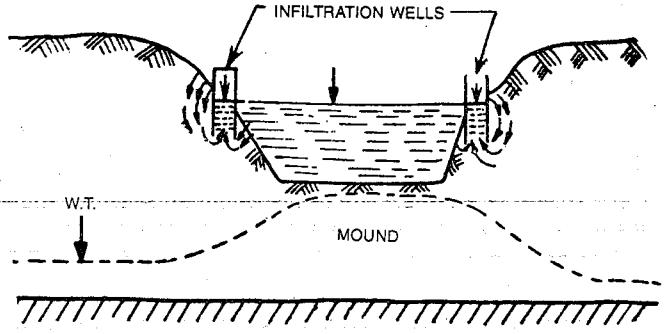


Fig. 16.31. Location of infiltration wells.

The various infiltration wells are connected by porous pipes to a *sump well*, called **jack well**, as shown in Fig. 16.33. The water reaching the jack well from different infiltration wells is lifted, treated and distributed to the consumers.

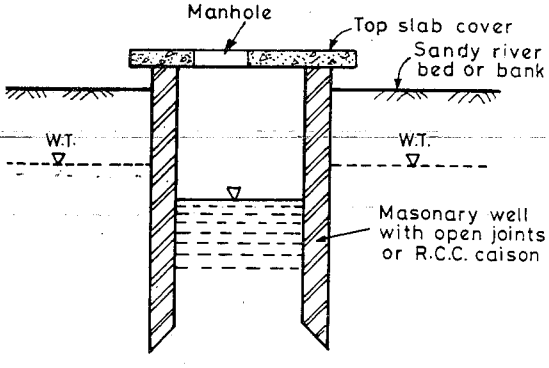


Fig. 16.32. Section of an infiltration well.

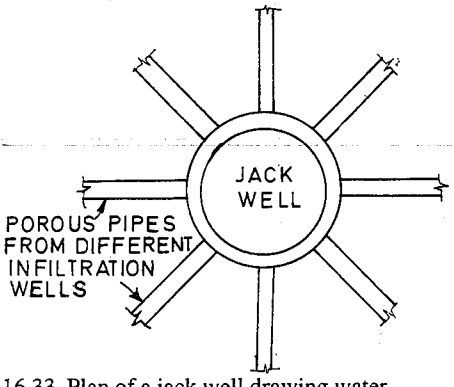


Fig. 16.33. Plan of a jack well drawing water from several infiltration wells.

Ranney Wells. A new technique that has recently started is to construct a vertical well of 3 to 6 m in diameter with horizontal radial collectors, and is known as a **radial well.** Such a well is sunk up to the required level and *plugged at the bottom.* Horizontal perforated steel pipes are then driven just at the level of the aquifer in the well by powerful hydraulic jacks. The length of these pipes or *radial collectors* may be of the order of 60 to 80 m or so. About 10 collectors can be installed at one level. Some other set or sets of such radial collectors can be installed at other levels, if possible, so as to increase the yield. The inner end of each collector pipe is fitted with a sluice valve which can be operated from the pump house above. The inflow of water into the well is thus controlled. The water from the well obtained by this method is generally clean, fresh and free from bacterial contamination. These wells are, therefore, very useful for drawing water from polluted streams. This type of well construction is very common in *France*, and is sometimes referred to as *French system of tapping underground water.*

A patented type of a radial well is known as a **ranney well**, or a ranney collector. It consists of an R.C.C. caisson of 4.3 m (13′) in diameter, 0.45 m (18") thick, which is sunk into the ground up to the required level, and from which radial collectors are projected, as explained above.

16.23. Springs

The natural outflow of ground water at the Earth's surface is said to form a *spring*. A pervious layer sandwiched between two impervious layers, give rise to a natural spring. A spring indicates the outcropping of the watertable.

The springs are generally capable of supplying very small amounts of water, and are, therefore, generally not regarded as sources of water supplies. However, good developed springs may sometimes be used as water supply sources for small towns, especially in hilly areas. Certain springs, sometimes discharge hot water due to the presence of sulphur in them. These **hot springs** (such as the one in Sohana in Haryana; a group on the bed and bank of Sutlej river at Tattapani near Simla ; and also another at Manikaran near Manali on Parvati river in H.P. state) usually emit sulphur mixed water (warm to boiling), and hence cannot be used for water supplies, though sometimes useful for taking dips for the cure of certain skin ailments.

16.23.1. Formation and Types of Springs. Springs are usually formed under three general conditions of geological formations, as explained below :

(*a*) **Gravity springs.** When the ground watertable rises high and the water overflows through the sides of a natural valley or a depression (as shown in Fig. 16.34), the spring formed is known as a gravity spring. The flow from such a spring is variable with the rise or fall of watertable.

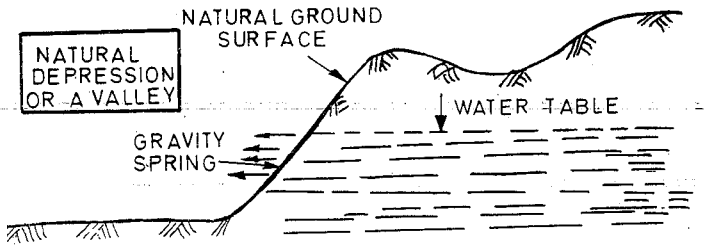


Fig. 16.34. Gravity spring.

(*b*) **Surface springs.** Sometimes, an impervious obstruction or stratum, supporting the underground storage, becomes inclined (such as shown in Fig. 16.35), causing the watertable

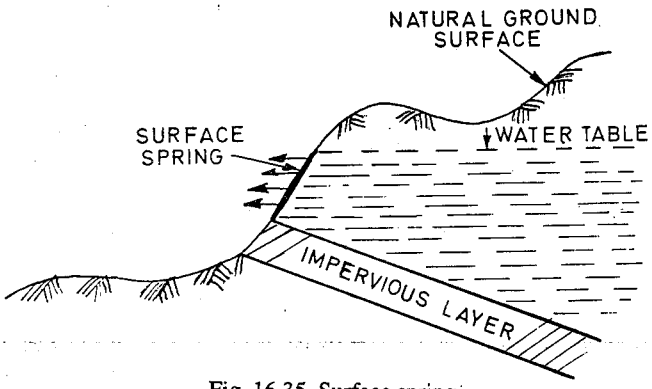


Fig. 16.35. Surface spring.

to go up and get exposed to the ground surface. This type of a spring is known as a *surface spring*. The quantity of water available from such springs is quite uncertain.

(*c*) **Artesian springs**. When the above storage is under pressure (*i.e.* the water is flowing through some confined aquifer), such as shown in Fig. 16.36, the spring formed is known as *artesian spring*. This type of springs are able to provide almost uniform quantity of water. Since the water oozes out under pressure, they are able to provide higher yields and may be thought of as the possible sources of water supply.

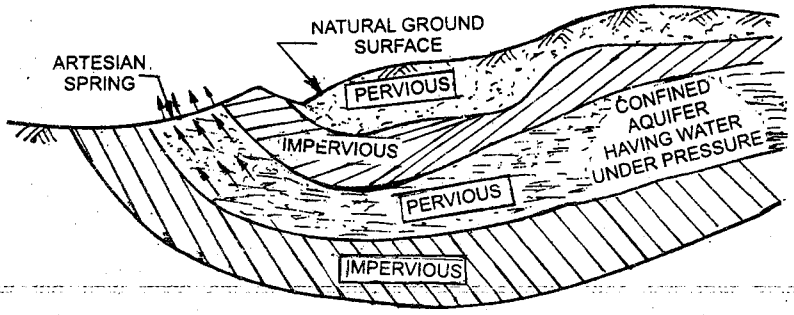


Fig. 16.36. Artesian spring.

When a spring issues out of the ground, a lake or a pond gets formed. Pucca masonry walls are, therefore, constructed on four sides of this tank or pond which is covered at the top. Proper cleaning of the area and arrangements for excluding the surface waters from entering the spring should be properly made, so as to avoid contamination of spring water. The water collected in the tank or the pond can be carried through pipes for meeting the water demand.

WELLS

A water well is a hole usually vertical, excavated in the Earth for bringing ground water to the surface. The wells may be classified into two types :

(1) Open wells ; and (2) Tube wells.

16.24. Open Wells or Dug Wells

Smaller amount of ground water has been utilised from the ancient times by open wells. Open wells are generally open masonry wells, having comparatively bigger diameters, and are suitable for low discharges of the order of 1—5 litres per second. The diameter of open wells generally vary from 2 to 9 m, and they are generally less than 20 m in depth. The walls of an open well may be built of precast concrete rings or in brick or stone masonry. Their thickness generally varies from 0.45 to 0.75 m, according to the depth of the well (See Fig. 16.37)

The yield of an open well is limited because such wells can be excavated only to a limited depth where the ground water storage is also limited.

Moreover, in such a well, the water can be withdrawn only at the critical velocity for the soil. Higher velocities cannot be permitted as that may lead to disturbance of soil grains and consequent subsidence of the well lining in the hollow so formed. The limit placed on the velocity, therefore, also limits the maximum possible safe discharge of an open well.

One of the recent methods used to improve the yield of an open well is to put in a 8 to 10 cm diameter bore hole in the centre of the well, so as to tap additional water from an aquifer or from the fissures in the rock. If a clay or a kankar layer is available at a smaller depth to support the open masonry well, a bore hole can be made in its centre so as to reach the sand strata. Such an arrangement will not only give a structural support to the open well but will also considerably increase its yield. Depending upon the availability of such a provision, the open wells may be classified into the following two types :

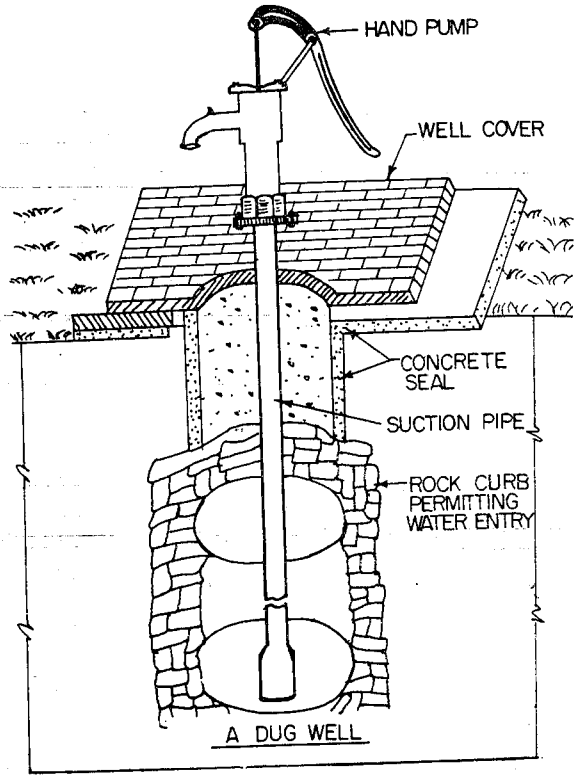


Fig. 16.37. Open well fitted with hand pump.

(*a*) *Shallow open wells* ; and (*b*) *Deep open wells.*

Shallow well is the one which rests in a pervious stratum and draws its supply from the surrounding material. On the other hand, a deep well is the one which rests on an impervious '*mota*' layer and draws its supply from the pervious formation lying below the mota layer, through a bore hole made into the '*mota*' layer, as shown in Fig. 16.38. The term "mota layer," also sometimes known as "Matbarwa" or "Magasan", refers to a layer of clay, cemented sand, kankar or other hard materials, which are often found lying a few metres below the watertable in the subsoil. These names are not applied to the layers of hard material lying

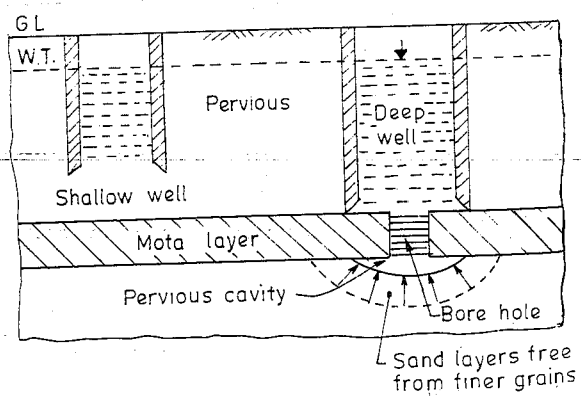


Fig. 16.38. Shallow and deep dug wells.

above the watertable. The main advantage of such a mota layer lies in giving structural support to the open well resting on its surface. It is useful for unlined and partly lined wells, and is indispensable for a heavy masonry well, which would not remain stable under steady use without such a support. The mota layer may either be continuous or may be localised, and are generally found in different thicknesses and depths at different places.

The nomenclature of shallow and deep dug wells is purely technical and has nothing to do with the actual depth of the well. A "shallow dug well" might be having more depth than a "deep dug well".

Since a shallow well draws water from the topmost water bearing stratum, its water is liable to be contaminated by the rain water percolating in the vicinity, which may take with it minerals or organic matter from decomposing animals and plants, etc. The water in a deep well, on the other hand, is not liable to get such impurities and infections. Secondly, the pervious formations below the mota layer generally contain greater quantities of ground water, yielding high specific yield. Hence, greater discharge and greater supplies can be obtained from a deep well as compared to those from a shallow well.

Water is generally drawn from dug or open wells by means of a bucket and a rope. However, due to the possible surface contamination of water in an uncovered well and also the individual buckets adding contamination to the water, such open wells have been covered in many parts of India and fitted with *hand pumps* (Fig. 16.37). A hand pump uses a reciprocating type of a pumping arrangement to lift water and is described in details in chapter 7 of "Water Supply engineering" by the same author.

16.24.1. Cavity Formation in Open Wells. Consider a well from which no water is being withdrawn. The water level in such a well will obviously be the same as is the static watertable outside the well. Now, if a discharge is withdrawn from this well at a constant rate, the level in the well will go down and stabilise at a lower level than that of the outside watertable. The head difference between these two levels is called the **depression head** (Fig. 16.39). Under the influence of this head difference, water enters the well from outside, so as to fill the gap created by the withdrawn water. As the water from the surrounding soil travels towards the well, there is a gradual loss of head, and water surface drops towards the well. Since the same discharge is passing through the reducing soil areas as it approaches the well, there is a gradual increase in the flow velocity towards the well. Now according to Darcy's law, this velocity can gradually increase only if the hydraulic gradient gets gradually increased. Hence, the water surface will fall gently in the beginning and will fall more and more rapidly as it approaches the well. The surface of watertable surrounding the well, therefore, takes up a curved shape and is called the **cone of depression**. At a certain distance from the well, there is no appreciable depression of watertable. This distance from the central line of the well is called the *radius of influence* of the well.

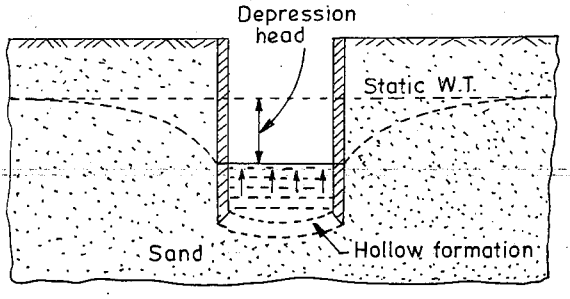


Fig. 16.39

The velocity of percolating water into the well depends upon the depression head. If more amount of water is withdrawn from the well and thereby increasing the depression head, higher flow velocities will prevail in the vicinity of the well. Thus, at a certain

rate of withdrawal, it is very much possible that the flow velocity may exceed the critical velocity for the soil, thereby causing the soil particles to lift up. As more and more sand particles are lifted, a hollow is created in the bottom of the well, resulting in the increased effective area, so that ultimately, the velocity falls below the critical value and then no further sand goes out of the well.

As pointed out earlier, the formation of such hollows beneath the wells are dangerous in shallow wells, because there is always a danger of subsidence of the well lining. *The maximum rate of withdrawal from such wells is therefore, limited.*

In case of a deep well resting on mota layer, the cavity or hollow formation below the bore hole (Fig. 16.38) is not dangerous, because the well lining remains supported on the mota layer. Hence, a hollow, much larger in area than the cross-sectional area of the well, may safely form in deep wells, and thereby giving higher yields. In a shallow well of an equivalent yield, the well area will have to be increased equal to the area of the cavity under the deep well, which would make it costlier.

16.24.2. Construction of Open Wells. From the construction point of view, the open wells may be classified into the following three types :

Type I. Wells with an impervious lining, such as masonry lining, and generally resting on a mota layer.

Type II. Wells with a pervious lining, such as the dry brick or stone lining, and fed through the pores in the lining.

Type III. No lining at all *i.e.*, a Kachha well.

All these three types of wells are discused below :

Type I. *Wells with impervious lining.* They provide the most stable and useful type of wells for obtaining water supplies. For constructing such a well, a pit is first of all excavated, generally by hand tools, up to the soft moist soil. Masonry lining is then built up on the *kerb* upto a few metres above the ground level. A "Kerb" is a circular ring of R.C.C., timber or steel having a cutting edge at the bottom and a flat top, wide enough to support the thickness of the well lining called "steining". The kerb is then descended into the pit by loading the masonry by sand bags, etc. As excavation proceeds below the kerb, the masonry sinks down. As the masonry sinks down, it may be corrected by adjusting the loads or by removing the soil from below the kerb which may be causing the tilt. The well lining (steining) is generally reinforced with vertical steel bars.

After the well has gone up to the watertable, further excavation and sinking may be done either by continuously removing the water through pumps, etc., or the excavation may be carried out from top by the *Jhams*. A *Jham* is a self-closing bucket which is tied to a rope and worked up and down over a pulley. When the *Jham* is thrown into the well, its jaws strike the bottom of the well, dislodging some of the soil mass. As the *Jham* is pulled up, the soil cuttings get retained but the water oozes out. The sinking is continued till the mota layer is reached. A smaller diameter bore hole is then made through the mota layer in the centre of the well, which is generally protected by a timber lining.

Sometimes, when mota layer is not available, shallow wells may be sunk as described above upto a required depth, and partly filled with gravel or broken ballast. This will function as a filter through which water will percolate and enter the well but the sand particles will be prevented from rising up.

In a pucca well, lined with an impervious lining on its sides, the flow is not radial. The water enters only from the bottom and the flow becomes spherical when once the cavity has been formed at the bottom.

Type II. *Wells with pervious lining.* In this type of wells, dry brick or stone lining is used on the sides of the well. No mortar or binding material is used. The water, thus enters from the sides through the pores in the lining. The flow is, therefore, radial. Such wells are generally plugged at the bottom by means of concrete. If the bottom is not plugged, the flow pattern will be a combination of radial flow and a spherical flow. *Such wells are generally suitable in strata as of gravel or coarse sand.* Such a well is constructed in finer soils, so as to prevent the entry of sand into the well along with the seeping water.

Type III. *Kachha wells.* These are temporary wells of very-shallow depths, and are generally constructed by cultivators for irrigation supplies in their fields. Such wells can be constructed in hard soils, where the well walls can stand vertically without any support. They can, therefore, be constructed only where the watertable is very near to the ground. Though they are very cheap and useful, yet they collapse after sometime, and may sometimes prove to be dangerous.

16.24.3. Yield of an Open Well. As discussed in article 16.9, the yield of an open well can be determined:

(*i*) by *estimating the velocity of the ground water* :

(*ii*) by performing pumping tests, such as *equilibrium pumping test* or *recuperating test.* Both these tests were briefly described in article 16.9, and are elaborated in more details in this article.

(*a*) **Equilibrium pumping test.** A pump is first of all installed, so as to draw sufficient supplies of water from the open well, and to cause heavy drawdown in its water level. The rate of pumping is then changed and so adjusted that the water level in the well becomes constant. In this condition of equilibrium, the rate of pumping will be equal to the rate of yield of the well at a particular drawdown. Knowing this yield say Q_1 at a certain known drawdown, say s_1, the yield (Q) at any given drawdown (s) can be evaluated as follows :

By Darcy's law $Q = K.I.A = K \cdot \frac{s}{L} \cdot A = \frac{K}{L} A.s.$

or $$Q = C.A.s \qquad ...(16.38)$$

where s is the depression head or the drawdown in the well.

If Q_1 is the known discharge at a certain known drawdown s_1, we have

$$Q_1 = C.A.s_1 \qquad ...(i)$$

Q_1 and s_1 are known and A is the area of cross-section of the well. In case a cavity is formed, the area A is taken as $\frac{4}{3}$ times the actual cross-sectional area of the bottom of the well. Knowing Q_1, A and s_1 in Eq. (*i*) above, the value of C can be calculated. Hence, the discharge Q at any other value of depression head (s) can be easily worked out.

From the above equation, it is also evident that the velocity increases with the depression head (s). The value of s, for which the velocity becomes equal to the critical value, is called the *critical depression head.* Generally, the depression head is kept equal to $\frac{1}{3}$ times the critical depression head; and such a head is known as the *working head.*

Hence, the maximum yield or critical yield corresponds to the critical depression head, and the maximum safe yield corresponds to the working head.

(*b*) **Recuperating test.** Although the pumping test gives accurate value of the safe yield, it sometimes becomes very difficult to adjust the rate of pumping, so as to keep the well water level constant. In such circumstances, recuperating test is adopted.

In this method, water is first of all drained from the well at a fast rate so as to cause sufficient drawdown. The pumping is then stopped. The water level in the well will start rising. The time taken by the water to come back to its normal level or some other measured level is then noted. The discharge can then be worked out as below :

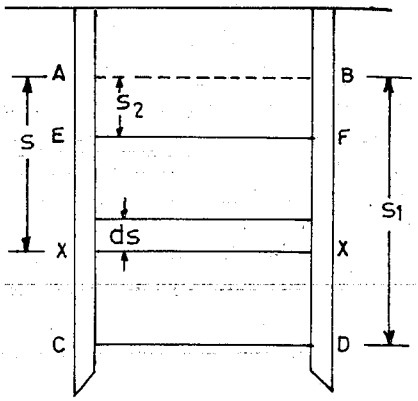


Fig. 16.40.

With reference to Fig. 16.40.

Let AB = Static water level in the well before the pumping was started

CD = Water level in the well when the pumping was stopped

s_1 = Depression head in the well at the time the pumping was stopped

EF = Water level in the well at the noted time (say after a time T from when the pumping is stopped)

s_2 = Depression head in the well at time T after the pumping is stopped.

Let X-X be the position of the water level at a time t after the pumping was stopped and let the corresponding depression head be s.

Let ds be the decrease in the depression head in a time dt after the time t.

Hence, in a time t after the pumping is stopped, the water level recuperates by $(s_1 - s)$. It again recuperates by ds in a time dt after this.

$\therefore$ Volume of water entering the well in the small interval of time (dt)

$$= dV = A.ds \qquad ...(1)$$

where A is the cross-sectional area of the well at the bottom.

Also, if Q is the rate of recharge into the well at the time t under a depression head s, then the volume of water entering the well in this small time interval is

$$= dV = Q.dt$$

But $Q \propto s$

$\therefore$ $$Q = C' . s \qquad ...(2)$$

when C' is a constant depending on the soil through which water enters the well.

$\therefore$ $$dV = C' . s . dt \qquad ...(3)$$

Equating (1) and (3), we get

$$-A.ds = C' .s.dt$$

(The –ve sign indicates that s decreases as t increases)

or $$\frac{C' .dt}{A} = -\left(\frac{ds}{s}\right)$$

Integrating between the limits

$$t = 0, \quad s = s_1, \quad t = T, \quad s = s_2$$

we get, $$\frac{C'}{A}\int_0^T dt = -\int_{s_1}^{s_2}\frac{ds}{s} \qquad \text{or} \qquad \frac{C'}{A}\cdot |t|_0^T = -\Big|\log_e s\Big|_{s_1}^{s_2}$$

or $$\frac{C'}{A}(T) = -\log_e\frac{s_2}{s_1} = -2.3\log_{10}\frac{s_2}{s_1} = 2.3\log_{10}\frac{s_1}{s_2}$$

$$\therefore \qquad \frac{C'}{A} = \frac{2.3}{\mathbf{T}}\log_{10}\frac{\mathbf{s_1}}{\mathbf{s_2}} \qquad \text{...(16.39)}$$

Knowing the values of s_1, s_2 and T from the above test, the value of $\frac{C'}{A}$ can be calculated. $\frac{C'}{A}$ is called the specific yield or the specific capacity of the open well in cumecs per sq-m of area under a unit depression head. Knowing the value of $\frac{C'}{A}$, the discharge Q for a well under a constant depression head s can be calculated as follows:

$$Q = C' \,.\, s$$

or $$Q = \left(\frac{C'}{A}\right)A.s \qquad \text{...(16.40)}$$

or $$Q = \left(\frac{2.3}{T}\log_{10}\frac{s_1}{s_2}\right)A.s \qquad \text{...(16.41)}$$

A and s are known, discharge for any amount of drawdown (s) can be easily worked out.

In the absence of recuperation test, the following rough values of $\frac{C'}{A}$, as given by Marriot, can be used.

Table 16.3

Type of soil	$\frac{C'}{A}$ *(i.e. specific capacity or yield) in cubic metres per hour per sq. metre of area under unit drawdown*
Clay	0.25
Fine sand	0.50
Coarse sand	1.00

(*iii*) The third method which is useful for determining the yields of open wells as well as those of tubewells involves the use of equations developed by Dupuit, Thiem, etc., as discussed in the previous articles.

Example 16.10. *Design an open well in coarse sand for a yield of 0.004 cumec when operated under a depression head of 3 metres.*

Solution.

The discharge required from the well

$$= 0.004 \text{ cumec} = 0.004 \text{ cubic metre per second}$$

$$= 0.004 \times 60 \times 60 \text{ cubic metres per hour} = 14.4 \text{ m}^3/\text{hr}.$$

From Table 16.3, the value of $\frac{C'}{A}$ for coarse sand may be taken as

$$1.0 \text{ m}^3/\text{hr}/\text{m}^2/\text{m of depression head.}$$

Also, Depression head $= s = 3$ m.

Now from Eq. (16.40) $Q = \left(\frac{C'}{A}\right) A.s$

$\therefore \quad 14.4 = 1 \times A \times 3$

or $\quad A = \frac{14.4}{3} = 4.8 \text{ m}^2.$

If d_w is the diameter of the well, then $\frac{\pi d_w^2}{4} = 4.8$

or $\quad d_w = \sqrt{\frac{4.8 \times 4}{\pi}} = 2.48 \text{ m}$; **say 2.5 m** **Ans.**

Example 16.11. *During a recuperation test, the water level in an open well was depressed by pumping by 2.5 metres and is recuperated by an amount of 1.6 metres in 70 minutes.*

(a) Determine the yield from a well of 3 m diameter under a depression head of 3.5 metres.

(b) Also determine the diameter of the well to yield 10 litres/second under a depression head of 2.5 metres.

Solution.

From Equation (16.39) $\frac{C'}{A} = \frac{2.3}{T} \log_{10} \frac{s_1}{s_2}$

where s_1 = Initial drawdown = 2.5 m

s_2 = Final drawdown = 2.5 − 1.6 = 0.9 m

T = Time = 70 minutes = 70 × 60 = 4,200 sec.

$\therefore \quad \frac{C'}{A} = \frac{2.3}{4{,}200} \log_{10} \frac{2.5}{0.9} = \frac{2.3}{4{,}200} \log_{10} 2.778 = 0.244 \times 10^{-3}$

(*a*) Yield from a well of 3 m diameter, under a depression head of 3.5 m, is obtained from equation (16.40) as

$$Q = \left(\frac{C'}{A}\right) A.s = 0.244 \times 10^{-3} \times \left(\frac{\pi}{4} \times 3^2\right) \times 3.5 = 6.02 \times 10^{-3} \text{ m}^3/\text{s}$$

$= 6.03$ litres/sec. **Ans.**

(*b*) If $Q = 10$ litres/sec

$s = 2.5$ m

$$Q = \left(\frac{C'}{A}\right) \cdot A \cdot s$$

$$10 \times 10^{-3} = (0.244 \times 10^{-3}) A \times 2.5$$

or $\quad A = \frac{10}{0.244 \times 2.5} = 16.4$ or $\frac{\pi}{4} d^2 = 16.4$

or $\quad d = 4.57$ m ; say 4.6 m

Hence, the diameter of the well required = 4.6 m **Ans.**

16.25. Tubewells

The discharge from an open well is generally limited to 3 to 6 litres/sec. Mechanical pumping of small discharges available in open wells is not economical. To obtain large discharge mechanically, *tubewell, which is a long pipe or a tube, is bored or drilled deep into the ground, intercepting one or more water bearing stratum.* The discharge of an open well is smaller, because : (*i*) open wells can tap only the topmost or at the most the next lower water bearing stratum. (*ii*) water from open wells can be withdrawn only at velocity equal to or smaller than the critical velocity for the soil, so as to avoid the danger of well subsidence. But in the tubewells, larger discharges can be obtained by getting a larger velocity as well as a

larger cross-sectional area of the water bearing stratum. Since, we have an enormous storage of ground water in India, the tubewells provide excellent method of providing water supplies, although they are generally used for irrigation.

16.25.1. Tube-wells in Alluvial Soils. Most of our land, especially the entire area from the Himalayas to the Vindhya mountains (such as the Indo-Gangetic plain), coastal areas, Narmada valley, etc., consist of deep alluvial soils. The subsoil water slowly penetrates and is stored in the porous sand and gravel beds which are extensively found in India, except that in the desert areas. Tubewells can be easily installed in such soils and are very useful for irrigation. It is in this context that the tubewells are assuming greater and greater importance for tapping our ground water resources, especially in alluviums.

Deep tubewells are as deep as 70 to 300 m, and tap more than one aquifer. They are usually constructed in our country by the State Governments and are called State tubewells. Such wells may yield as high as 200 to 220 litres/sec. The general average yield from the standard tubewells is however of the order of 40 to 45 litres/sec. A 300 m deep tubewell has been constructed at Allahabad (U.P.) at the edge of the river Ganga and is yielding at about 140 litres/sec. The diameter of the hole is 0.6 m upto 60 m depth and then 0.56 m below 60 m. The diameter of the strainer is 0.25 m and drawdown is 10 m. Such deep tubewells are drilled by *heavy duty rotary drilling rigs* (direct rotary as well as reverse rotary). *Percussion drilling rigs* may also be used in hard boulder areas. There exist about 50,000 deep tubewells in our country, and every year about 1,000 such wells are being added. Most of our deep tubewells have been constructed by using mild steel slotted pipe screens with gravel packing.

Besides the deep tubewells, **shallow tubewells**, having 20 to 70m depth and tapping only one aquifer, are also constructed, usually by private individual cultivators. Such wells may yield as high as 15–20 litres/sec, if located at proper places. Each well irrigates about 8 hectares. Such shallow tubewells are usually drilled by *light rigs, cable tool drills, water jet methods*, and *hand boring devices*. There exist about 50 lakh shallow tubewells in India, and every year about 2.5 lakh such wells are being added.

16.25.2. Tubewells in Hard Rocky Soils. It is very difficult to construct a tubewell irrigation system in rocky areas. Therefore, in rocky areas, tubewells are resorted to only when, there are no other alternate sources of water. Hence, in rocky areas, only isolated holes of 10 to 15 cm diameter are drilled using *down the hole hammer rigs* (DTH rigs). They are usually in depth range of 100m, although tubewells up to 300 m depths have been successfully bored. Such tubewells are called **bored wells** because the bore hole is able to stand on its own in the bottom portion, and a tube is pushed only in the upper weathered zone. These wells usually depend on joints & fissures in the rocks for their water supply. Even with a heavy drawdown of 20 to 30 m, such wells are generally not able to yield more than 5–10 litres/sec, except when they tap some embedded aquifer. Such tubewells have mostly been constructed in southern States of our country.

16.25.3. Types of Tubewells. Depending upon the entry of the water through a cavity or a screen, the tubewells can be broadly classified into the following two categories :

(1) *Cavity type tubewells;* and (2) *Screen type tubewells.*

Both these types of tubewells are discussed below:

(1) **Cavity Type Tubewells.** A cavity type tubewell draws water from the bottom of the well, and not from the sides, as is done by a screen well. The flow in a cavity well, therefore, is essentially spherical, and not radial like that of a screen well.

Moreover, since in a gravity well, the water is drawn from the bottom of the well, such a well can tap only one water bearing stratum. Such a tubewell however, is very economical, as it requires only plain well pipe, which is lowered into the bore made through the ground strata up to the required depth, so as to tap the requisite aquifer.

The principle behind the working of a cavity-type tubewell is essentially similar to that of a deep open well, with the only difference that whereas an open deep well taps the first aquifer, just below the mota layer, a cavity tubewell need not do so, and may even tap the lower or still lower stratum, as shown in Fig. 16.41

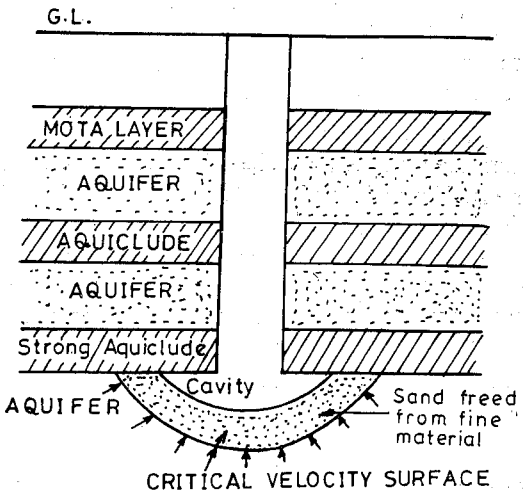


Fig. 16.41. Cavity type tubewell.

A cavity type tubewell essentially consists of a pipe bored through the soil and resting on the bottom of a strong clay layer. A cavity is formed at the bottom, and the water from the aquifer enters the well pipe through this cavity, as shown in Fig. 16.41. In the initial stage of pumping, fine sand comes out with water and consequently a hollow or a cavity is formed. As the spherical area of the cavity increases outwards, the radial critical velocity decreases for the same discharge, thus reducing the flow velocity and consequently stopping the entry of sand. Hence, the flow in the beginning is sandy but becomes clean with the passage of time.

The cavity of such a tubewell, however needs to be **developed** carefully and slowly by using a *centrifugal pump* rather than a *compressor* or a *turbine pump.*

To begin with, the water is pumped from the well at a low discharge rate. When the discharging water becomes clear, the drawdown may be increased slightly, which may result in further sand being drawn out. The process is repeated till the normal drawdown and clear discharge is obtained. The pumping is then stopped for an hour or so, and then resumed again. The discharge after restarting may again contain sand. The pumping is continued till the water is clear again. The procedure may be repeated till the tubewell is fully developed, which is shown by the sand free discharge coming out even on the resumption of pumping after a closing interval.

Cavity type tubewells can however be used only for small supplies, particularly for domestic supplies.

(2) **Screen Type Tubewells.** Screen type tubewells are most widely adopted and have been extensively constructed in our country particularly for irrigation purposes. So much so that whenever we refer to a tubewell, we generally mean a "screen well". All the State tubewells constructed in U.P. from where the technique of tubewell construction started in 1931, are exclusively of this type. Such a well can easily tap a number of aquifers, and hence does not depend only on one aquifer, like a cavity well. Screen type tubewells can be further divided into the following two types :

(*i*) *strainer tubewells;* and (*ii*) *slotted pipe gravel-pack tubewells.*

(*i*) A **strainer tubewell** uses strainer lengths lowered into the bore hole and located opposite the water bearing formations, whereas, plain pipe lengths are located opposite the non water bearing formations, as shown in Fig 16.42. A bail plug is provided at the bottom. Water enters into the well through these strainers from the sides, and the flow is *radial.*

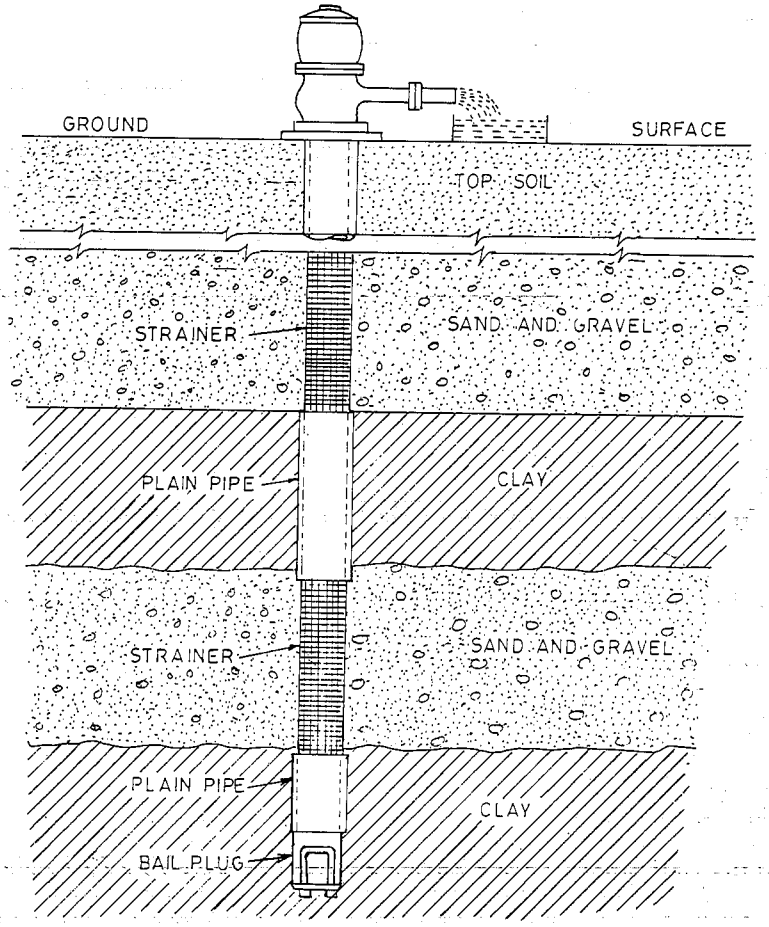


Fig. 16.42. A strainer tubewell.

A **strainer** (Fig 16.43) essentially *consists of a perforated or a slotted pipe with a wire mesh wrapped round the pipe with a small annular space between the two.* The wire screen prevents the sand particles from entering the well. The water, therefore, enters the well pipe through the fine mesh and the particles of size larger than the size of the mesh are prevented from entering the well. This reduces the danger of sand removal, and hence larger flow velocities can be permitted.

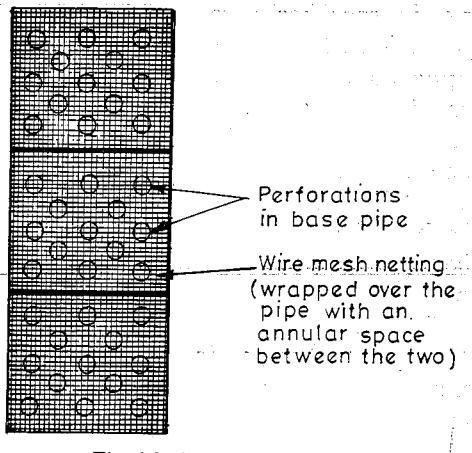


Fig. 16.43. A typical strainer.

The perforated pipe is made to have the cross-sectional area of its openings equal to that in the wire mesh, so that no change of velocity occurs between the two. An annular space between the pipe and the wire mesh is certainly required, otherwise the wires of the mesh will cover a large part of the area of the pipe openings.

A strainer type tubewell is generally unsuitable for fine sandy strata, because in that case, the size of the mesh openings will have to be considerably reduced, which may result in choking of the strainer; and if the mesh openings are kept bigger, the well will start discharging sand.

(*ii*) A **slotted pipe gravel pack tubewell**, on the other hand, uses a *slotted pipe without being covered by any wire mesh.* Such slotted pipe lengths are located opposite the water bearing formations, as is done with the strainers in a strainer tubewell. These slotted pipes should strictly be referred to as "screen pipes", although, however, they are often called as "*slotted pipe strainers*" or "*pipe strainers*", thereby having little scope to differentiate between such slotted pipes (*i.e. screen pipes*) and the perforated pipes with mesh coverings (*i.e. strainers*).

After placing the assembly of the plain and slotted pipes in the bore hole, a mixture of gravel and bajri (called gravel shrouding) is poured into the bore hole between the well pipe assembly and the casing pipe, so as to surround the well pipes by a designed optimum thickness of gravel pack. The gravel packing is specifically required around the screen pipes, but since the material is poured from the ground level into the bore hole, the pack shall be installed in the entire depth of the well below the top level of the shallowest screen, as shown in Fig. 1644. Sometimes, the gravel pack is provided even above the top level of the shallowest screen, and up to the ground level, so as to obtain a stable and an efficient tubewell.

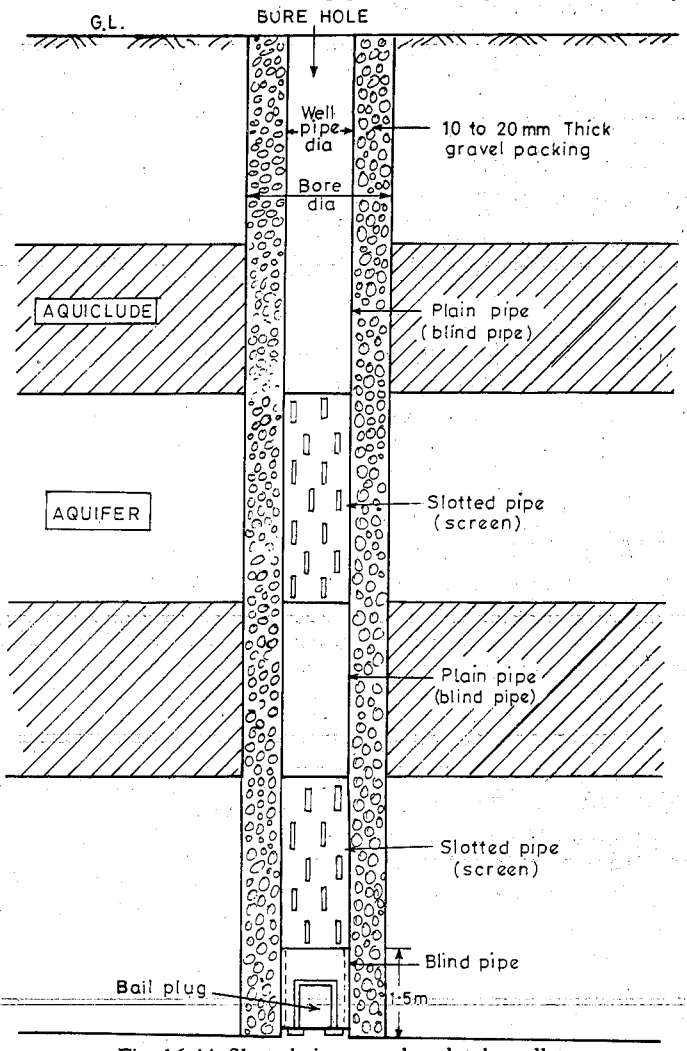


Fig. 16.44. Slotted pipe gravel pack tubewell.

Gravel pack wells are generally provided in fine aquifers, where the effective grain size (D_{10}) may be less than 0.25 mm, and uniformity coefficient (C_u) may be 2.0 or less. *Gravel packing also enables the use of single size slots in the screen pipes, irrespec-*

tive of the fact that fine as well as coarser aquifers are tapped simultaneously and in alternating layers. Gravel pack is therefore highly preferred for deep tubewells, which tap more than one aquifer. Due to this reason, most of the State deep tubewells have been constructed in our country in fine alluvial soils, by providing cast in situ gravel packs, throughout the full height of the wells.

Precast gravel packs have also come into the market, where the slotted pipes are mounted with *prepack gravel filters*, popularly called *Ashim filters*. In these filters, graded hard gravel grains are coated and bonded together with water proof chemicals over the outer surface of the slotted pipe, providing a highly permeable screen. Such prepack gravel filters are found suitable for shallow and medium deep tubewells in aquifers composed of fine sand, coarse sand and gravel. Prepacking is claimed to be providing economy in the quantity of gravel used and protects against dislodging of the gravel envelope. However, such filters are so far not being used in larger projects in our country.

16.25.4. Design of a Well Screen. A well screen constitute the most important part of a tubewell, since it serves as the intake structure for the entry of water into the well. The design of the screen is largely influenced by the characteristics of the water bearing formations. Dry sieve analysis of the aquifer sample obtained during drilling of the bore hole, is therefore carried out to plot the grain size distribution curve for the aquifer. The results will help us to determine the design specifications of the well screen as well as to design the gravel pack.

From the grain size distribution curve of the aquifer, the following characteristics of the aquifer are determined :

(*i*) Effective size (D_{10})

(*ii*) Uniformity Coefficient (C_u)

(*iii*) D_{50} size (mean particle size) of the aquifer.

The *effective size* (D_{10}) is that sieve size through which 10% of the particles shall pass, and 90% are retained. 90% of the particles in the aquifer are thus coarser than this size, and hence retained on the sieve of this size.

The *uniformity coefficient* (C_u) is defined as the ratio of the sieve size passing 60% of the aquifer material, to the sieve size passing 10% of the aquifer material. In other words,

$$C_u = \frac{D_{60}\,(40\%\,\text{retained})}{D_{10}\,(90\%\,\text{retained})} \qquad ...(16.42)$$

This ratio was proposed by Hazen (1893) to be a quantitative expression of the degree of the assortment of the water bearing sand, as an indicator of porosity. The value of C_u for complete assortment (one grain size) is 1; while for fairly even grained sand, it ranges between 2 to 3. For heterogeneous sand, the value will be higher. *Generally a material is classified as uniform, if its uniformity coefficient C_u is equal to or less than 2.*

Designing the Size of the Slot Openings of a Screen. Fixing an optimum size for the slot openings of a screen is an important component of the screen design, and it depends upon the size of the aquifer material. Oversized slots will pump finer material indefinitely, and it will be difficult to obtain clear water. Undersized slots will provide more resistance to the flow of ground water into the well, resulting in more head loss and corrosion. Fine slots are also blocked by small sand and silt particles. The problem of clogging is reduced as the size of the openings is increased. *Therefore, theoretically, the slot openings should be as wide as possible.* The optimum value is determined by matching the size of the opening

with the grain size distribution of the material surrounding the screen. *In practice, the slot size varies from values as low as 0.2 mm to as large as 5 mm.* In India, two sizes *i.e.* 1.6 mm and 3.2 mm are generally available in the market. The criteria for selecting the size of the slot openings for non-gravel pack and gravel pack well is given below :

(*a*) **Slot openings for non-gravel pack wells.** The optimum slot opening size is chosen as the one which retains 40% of the sand (*i.e.* equal to D_{60} size of the aquifer) if the ground water is non-corrosive; and equal to the D_{50} size of the aquifer if the ground water is corrosive.

The above optimum selected slot size may vary for different aquifers, when more than one aquifer is to be tapped, thereby indicating the use of different widths for slots to be made in the pipe lengths to be rested against the different aquifers, Practically this is a difficult proposition, and hence uniform width of slots designed on the basis of the finest aquifer material are often provided.

The above adoption of D_{60} size for the slots indicates that 60% of the formation material shall be pumped out during the development of the well, which will result in the removal of all the fine particles from around the well screen.

(*b*) **Slot openings for gravel pack wells.** The slot size for well screens for gravel pack wells is determined on the basis of the grain size distribution curve of the gravel material used for the gravel pack. On this curve, a point is located indicating the 90% size of the gravel to be retained (*i.e.* D_{10} size for the gravel material), which indicates the optimum slot size for the well screen. The actual size of the slots may be fixed within ± 8% of the D_{10} size of the gravel pack.

Gravel pack design. The gravel pack should however also be designed before designing the size of the slots to be made in the well screens. The gravel pack is usually designed on the basis of **Pack Aquifer ratio** (PA ratio), which is usually defined as the ratio of D_{50} size of the gravel pack material to the D_{50} size of the aquifer material.

$$\therefore \quad \text{PA ratio} = \frac{D_{50} \text{ of the gravel}}{D_{50} \text{ of the aquifer}} \qquad ...(16.43)$$

Several recommendations have been made by different investigators for optimum values of PA ratio. The Central Board of Irrigation and Power (1967) has for Indian conditions, suggested the following criteria for PA ratio.

(*i*) PA ratio should be between 9 and 12.5 for uniform aquifers having $C_u \leq 2.0$ ...(16.44)

(*ii*) PA ratio should be between 12 and 15.5 for graded aquifers having $C_u > 2.0$ ...(16.45)

The uniformity coefficient of the gravel material is also preferably kept to be equal to or less than 2.5, because a higher value shall cause seggregation of the gravel shrouding during pouring, which will result in poor efficiency of operation of the well. Such gravel materials, having $C_u \leq 2.5$ are difficult to obtain, and hence gravel pouring requires a lot of field control. A numerical example on the design of the gravel pack has been solved in the next article to make its design very clear.

The *thickness of the gravel pack* should normally be fixed from practical considerations at about 7.5 cm, and in no case should exceed about 20 cm. A thicker envelope does not materially increase the well yield, nor will it reduce the possibility of sand pumping, because the controlling factor is the ratio of the grain size of the pack material and that of the aquifer. Too thick a gravel pack, instead of giving any advantages, may make the final development of the well more difficult.

Example 16.12. *The results of sieve analysis test carried out on a 500 gm sample of underground aquifer, proposed to be tapped for installation of a tubewell, are given in the table below :*

Size of the sieve in mm	*Mass of material retained in gm*
> 2.54	0.0
1.80	6.0
0.30	15.0
0.25	320.0
0.21	5.0
0.16	50.0
0.12	34.0
< 0.12	70.0
	500.0

Design the size of the gravel pack, and the slot size for the slotted screen pipes.

Solution. The given sieve analysis data is analysed, as shown in table 16.4. A distribution curve is then plotted between grain size in mm (on log x-axis) and % age finer (on y-axis), as shown in Fig. 16.45.

Table 16.4

Size of the sieve in mm	*Mass of material retained in gm*	*% age of material retained* $\frac{(2)}{500} \times 100$	*Cummulative % age retained*	*% age finer (p) 100 – col (4)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
> 2.54	0	0.0	0	100
1.80	6	1.2	1.2	98.8
0.30	15	3.0	4.2	95.8
0.25	320	64.0	68.2	31.8
0.21	5	1.0	69.2	30.8
0.16	50	10.0	79.2	20.8
0.12	34	6.8	86.0	14.0
< 0.12	70	14.0	100.0	0.0
Σ	500 gm	100%		

From the drawn curve, the following characteristics of the aquifer material are read out as :

$$D_{60} = 0.27$$
$$D_{50} = 0.265$$
$$D_{10} = 0.102$$

$$\therefore \quad C_u = \frac{D_{60}}{D_{10}} = \frac{0.27}{0.102} = \mathbf{2.64}$$

Since $C_u \geq 2.0$, we should use P.A. ratio for designing the gravel pack lying between 12 and 15.5

$$\text{Hence, P.A. ratio} = \frac{D_{50} \text{ of gravel pack}}{D_{50} \text{ of aquifer}} = 12 \text{ and } 15.5$$

$$\therefore \quad 12 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$\therefore$ D_{50} of gravel pack $= 12 \times 0.265 = \mathbf{3.18\ mm}$

$$\text{Also} \quad 15.5 = \frac{D_{50} \text{ of gravel pack}}{0.265}$$

$\therefore$ D_{50} of gravel pack $= 15.5 \times 0.265 = \mathbf{4.11\ mm}$

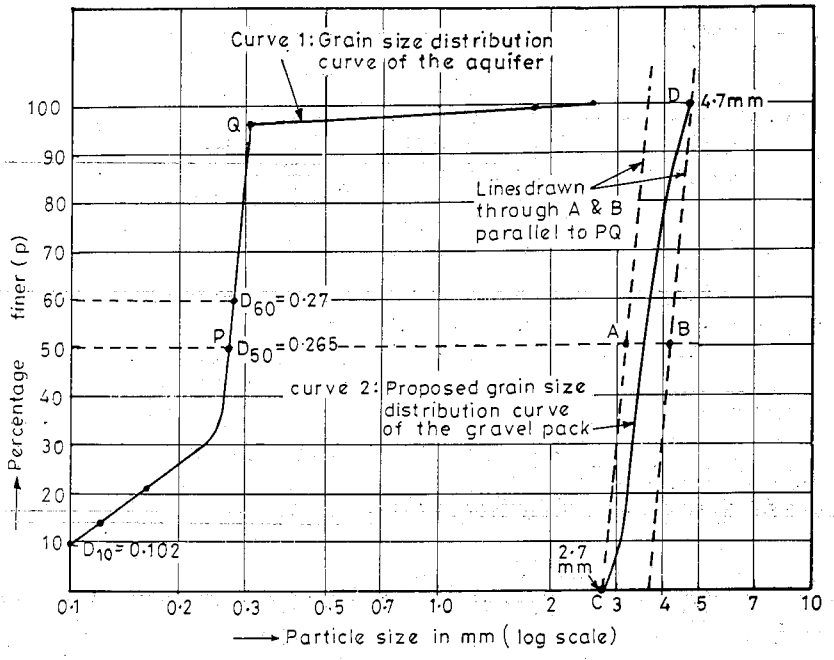


Fig. 16.45. Grain size distribution curves for the aquifer and the gravel pack.

In other words, D_{50} of the gravel pack should lie between these two limiting values; *i.e.* 3.18 mm and 4.11 mm. These values are marked on 50% horizontal line, as *A* and *B*. Lines (dotted) are drawn through these two points, parallel to the central portion of the grain size curve of the aquifer material. These two enveloping curves are the limiting curves for the grain size distribution curve of the gravel pack. The minimum size of the gravel between these two curves is represented by point *C* as 2.7 mm; and the maximum size is represented by point *D* as 4.7 mm. *Hence, the gravel pack size should vary between 2.7 mm and 4.7 mm.* The gravel should be screened such that the gravel size ranges between 2.7 mm and 4.7 mm. The proposed grain size curve for the gravel shall be somewhat as shown by the firm line *CD*. D_{10} size of this curve is approximately equal to 3 mm; and hence the screen having standard 3.2 mm wide slots may be used.

C_u for the gravel is also computed as $= \dfrac{D_{60}}{D_{10}} = \dfrac{3.6\text{ mm}}{3\text{ mm}} = 1.2$ (O.K.). This value should be ≤ 2.5 for good design and the drawn curve may be adjusted between the two envelope curves to achieve satisfactory value of C_u.

Designing the length and size of the screen. The total length of the screen to be provided for a tubewell shall be primarily controlled by the available thickness of the aquifers, since this length cannot exceed the aquifers thickness. It shall further be governed by the *area of the screen openings,* because to pass a given discharge, the screen length will be less if the area of its openings per m length is more, and *vice-versa.*

The area of the screen openings per m length of the screen usually veries between 15 to 20% of the screen area, which equals $\pi.d$, where d is the dia of the screen pipe. If the area of the openings is kept more than about 15 to 20 %, the structural strength of the well screen reduces, which consequently reduces the life of the screen.

The diameter of the screen is selected to satisfy the essential basic requirement that sufficient open area be provided in the screen, so as to limit the entrance velocity to a

safe permissible limit. The optimum safe entrance velocity for a given aquifer, is related to the coefficient of permeability (K) of the aquifer, as shown in Table 16.5.

Table 16.5 Optimum Screen Entrance Velocity (v_e)

Coefficient of Permeability (K) of the aquifer in cm/s	*Optimum entrance velocity (v_e) in cm/s*
0.02	1.5
0.05	2.0
0.09	3.0
0.14	4.0
0.18	4.5
0.24	5.0
0.28	5.5

For most of the available sandy aquifers, K value varies between 0.05 to 0.10 cm/s, and hence v_e generally varies between 2 cm/s to 3 cm/s.

When entrance velocity exceeds this safe optimum value, the frictional loss through the screen openings shall become higher, and there will be more incrustation and corrosion.

The entrance velocity is therefore calculated by dividing the design discharge per m length of the screen by the total area of the openings in the screen per m length of the screen. If the value works out to be more than the safe optimum value, the dia of the screen is increased, so as to increase the open area to achieve the optimum value of the entrance velocity. The guiding values of the screen diameter for different discharges, as suggested by USBR (Ahrens, 1970) are given in Table 16.6.

Table 16.6 Recommended Values of Screen Diameter

Discharge in l/min	*Recommended screen diameter in cm*
0 to 475	10
475 to 1125	15
1125 to 3000	25
3000 to 5250	30
5250 to 9500	35
9500 to 13,300	40

The minimum length of the screen can finally be designed by using the equation

$$\text{Min. length of the screen} = h = \frac{Q}{A_0 \cdot v_e} \qquad \text{...(16.46)}$$

where Q = Design discharge of the tubewell

A_0 = Area of the screen openings per m length of the screen

$= \pi.d \times$ percent of openings (p)

v_e = optimum entrance velocity for the given K value of the aquifer for non-gravel wells. For gravel pack wells, the mean K value for the aquifer and for the gravel pack is considered, to assume the value of v_e.

The above calculated minimum strainer length should be adjusted within the available aquifer depths by screening the available depth(s) by about 75 to 90% for confined aquifers. The percentage should be increased with the increase in thickness. Say for example, 75% screening is satisfactory for 8 m thick aquifers, and 90% for

20 m thick aquifers. Atleast 0.3 m aquifer depth at top as well as at bottom of the aquifer should be left unscreened to safeguard against the error in the placement of the screen during installation. The pumping water level should never fall below the top of the aquifer. The screen is usually located at the centre of the aquifer.

In unconfined aquifers, the bottom $\frac{1}{3}$ to $\frac{1}{2}$ depth of the aquifer is usually screened.

Example 16.13. *A fully penetrating well in a confined sandy aquifer has a maximum discharge capacity of 1200 l/min. The aquifer is overlain and underlain by impervious formations. The thickness of the aquifer is 20m. Design the length of the well screen assuming the percentage of the open area of the available strainer to be 15%, and bore hole dia as 15 cm.*

Solution. $Q = 1200\,\text{l/min} = \frac{1.2}{60}\,\text{m}^3/\text{s} = 0.02\,\text{m}^3/\text{s}$

A_0 = Area of the openings per m length of the screen

$= (\pi \cdot d)\, p = \pi\,(0.15\,\text{m}) \times 15\%$

$= \pi \times 0.15 \times 0.15\,\text{m}^2/\text{m} = \mathbf{0.071}\,\text{m}^2/\text{m}$ length of the screen

v_e = safe entrance velocity

= may be assumed to be 2 cm/s since K for sand is usually 0.04 cm/s

$\therefore \quad v_e = 0.02$ m/s

Using eqn. (16.46), we have

$$h = \text{Min. length of the screen} = \frac{Q}{A_0 \cdot v_e}$$

$$= \frac{0.02}{0.071 \times 0.02} = 14.2\,;\ \mathbf{15\ m}\ \text{(say)}$$

Since the aquifer thickness is 20m, and the minimum required screen length is 15m, it would be prudent to use 18 m length of the screen, which shall be about 90%* of the aquifer depth. The screen may be provided in the central 18 m depth of the aquifer, leaving one metre depth of the aquifer unscreened at both the ends.

16.25.5. Types of Well Screens. (1) **Slotted Pipe Screens.** As stated earlier, all the gravel pack wells normally utilise *slotted pipe screens*. The slot size is also decided somewhere near the D_{10} size of the designed gravel material. The slotted pipes to be used are generally made of mild steel. IS:8110-1976 provides details in respect of such screens. The standard slot size commonly adopted in India is 1.6 mm or 3.2 mm wide, and 10 cm long. The minimum spacing between slots is 3 mm. The slots are so arranged as to obtain an even distribution of flow over the entire periphery of the screen. They are distributed in groups of 3 or 4, and arranged so that the slots of one group are not in line with, those of the adjacent row, so as to maintain adequate strength in the well pipe. A typical view of the arrangement of slots is shown in Fig 16.46

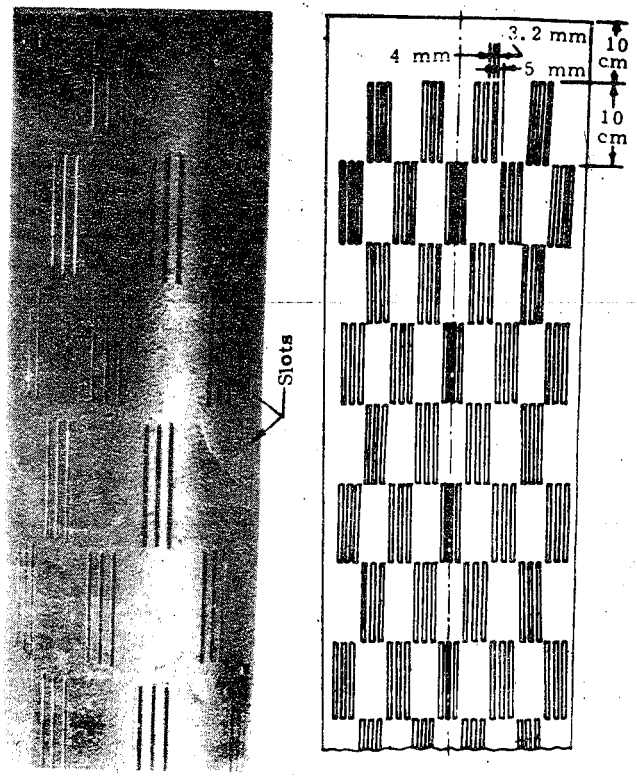
The slotted pipe is threaded at both ends. The bottom end of the slotted pipe is fitted with a blind pipe of 1.25m length, with a cap called *bail plug* at the bottom. The bail plug has an eye, fixed inside, which facilities the extraction of the tubewell assembly incase of failure.

Slotted pipe screens with prepack filters called **Ashim filters**, have also come into the market, and may be used for comparatively smaller and medium depth wells. A

* Confined aquifers are screened by about 75–90% of the thickness, whereas unconfined aquifers are screened in their bottom 1/3 to 1/2 of the thickness. Percentage of the confined aquifer to be screened increases with the increase in the thickness.

photographic view of such filters is shown in Fig 16.47. In such screens, mild steel slotted pipes are coated with 15–20 mm thick precast gravel layers, as to provide highly permeable screens. The mild steel pipe is usually slotted to have an open area of 25% with slots of 3.2 mm width. Different grades of filters are available to suit different sized aquifer particles. Such screens are, however, very costly, and hence adopted only for smaller individual wells.

Brass screens and **stainless steel screens**, utilising brass or stainless steel pipes, with slots as in mild steel pipes, are also available in the market. These screens are quite costly but are more durable and less liable to corrosion, incrustation and consequent contamination of water due



(*a*) General view (*b*) Layout of slots as per IS:8110-1976

Fig. 16.46. Mild steel slotted pipe screen.

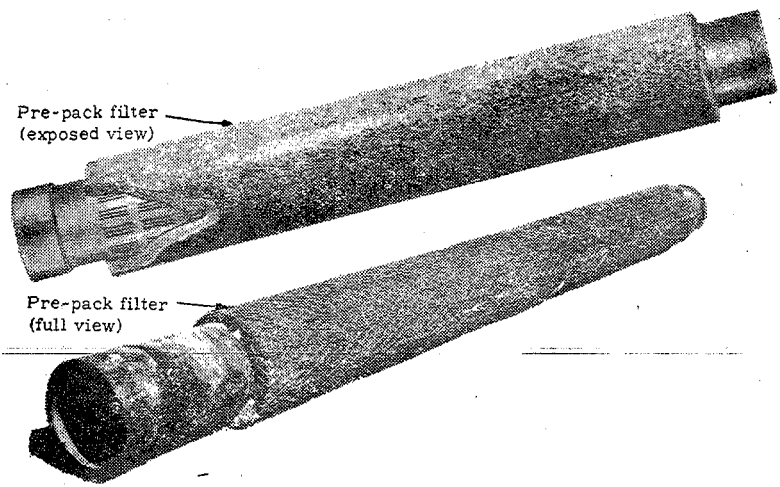


Fig. 16.47. Ashim's slotted pipe prepack filter screens.

to rusting. Hence, such screens may be preferred for tubewells to be used for water supplies. Such screens are manufactured from 4 mm thick copper or stainless steel sheets, provided with designed openings (slots), which is rolled and welded to make the

circular tube. The ends of the tube length (unslotted) are provided with outer threads for joining the tubes through a socket. Screens with different slot sizes and diameters are available in the market to suit the requirement of the design based on grain size distribution curve of the aquifer or the gravel pack.

(2) Strainer Type of Well Screens. Aquifers have traditionally been screened by utilising the strainer type of well screens. Such screens contain very small & narrow openings, to exclude the removal of those aquifer particles which are larger than the screen openings. Such screens usually have a double system of openings, consisting of slotted or perforated cylindrical pipes or shells, which are covered by fine wire mesh, wound round the pipe. The water has to initially enter the openings of the *outer wire jacket*, and then pass through the openings of the pipe shell. *Such strainer wells are usually provided without gravel packs, although it is not an absolute necessity, and even stainer wells can be gravel packed, like those using slotted pipes.*

Several types of strainers, which are generally used in tubewell construction, are:

(*i*) *Continuous-slot type of strainers.*

(*ii*) *Louver or shutter type of strainers;*

(*iii*) *pipe strainers with a cover of wire jacket, like that used in an agricultural strainer;* and

(*iv*) *Coir rope strainers.*

These types are briefly discussed below;

(*i*) **Continuous slot type of strainers** consist of cold drawn wire wound round a suitable cylindrical frame made of rods of iron, steel, brass etc; At each point where the wire crosses the rods, the two are welded and jointed to produce a one piece rigid unit. In order to avoid clogging, V-shaped openings are usually provided by using triangular shaped wire in the outer jacket. Such screens can be made practically in any width from 1.5 mm onward.

The well loss in such strainers is much-lower than those in the pipe strainers, but these are much costlier. A photographic view of such a strainer is shown in Fig. 16.48.

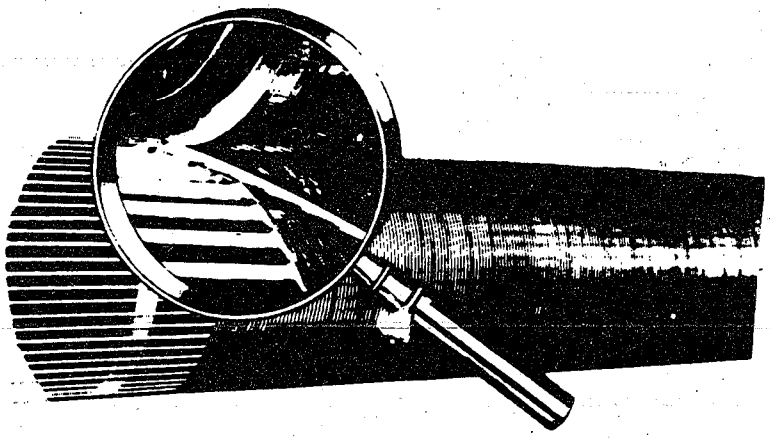


Fig. 16.48. Photoview of a continuous slot type of well strainer.

(*ii*) **Louver type of stainers** have openings, which are rows of louvers. The openings may be oriented either at right angles or parallel to the axis of the screen. The openings are produced in the wall of the welded tube by stamping process, using a die. A photo view of such a screen is shown in Fig 16.49. Such a screen provides lesser choice of the opening sizes, because sizes of dies are generally limited. Such a screen also provides lesser percent-

age of the open area than the one provided by the continuous slot type. The openings of such a screen also tend to clog during the development of the well, where the aquifer contains appreciable quantity of sand. There screens are, however, best suited when provided with gravel pack, as is done with the slotted pipe screens.

The continuous slot type of strainers, as well as the louver type of strainers, though are hydraulically more efficient, but have not become popular in developing countries like India, owning to their high costs.

(*iii*) **Pipe strainers with outer fine mesh jackets,** utilise perforated metal or pvc pipes surrounded by fine wire mesh, with some annular space between the two. Several designs are available in the market, and the most commonly adopted type is known as the **agricultural strainer.** A photoview of such a strainer is shown in Fig 16.50. It consists of a perforated galvanised iron pipe, on which steel or iron rods or strips of 1 cm width and 3mm thickness are welded at fixed intervals around the circumference of the pipe. The perforations are circular and made with a drilling machine. The perforations are arranged in such a manner, as not to be covered by the rods. Trapezoidal shaped bronze, brass or copper wire is wound round the series of rods, and securely shouldered, to provide a jacket of non-corrosive alloys.

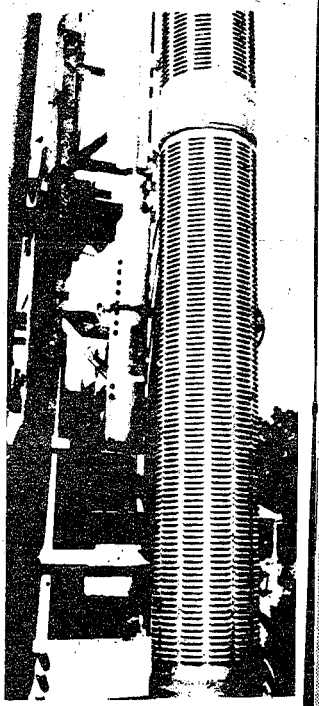


Fig. 16.49. Photoview of a louver type of well screen.

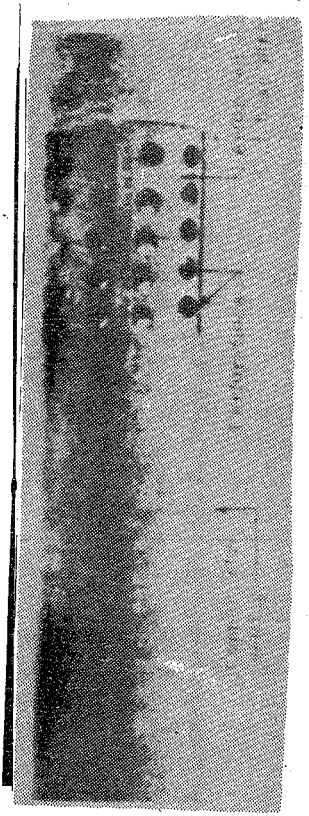


Fig. 16.50. Partially exposed photoview of an agricultural strainer.

The type of wire mesh to be used depends upon the aquifer particle size. Usually, three types of wire nettings suitable for fine, medium and coarse aquifers are available in the market.

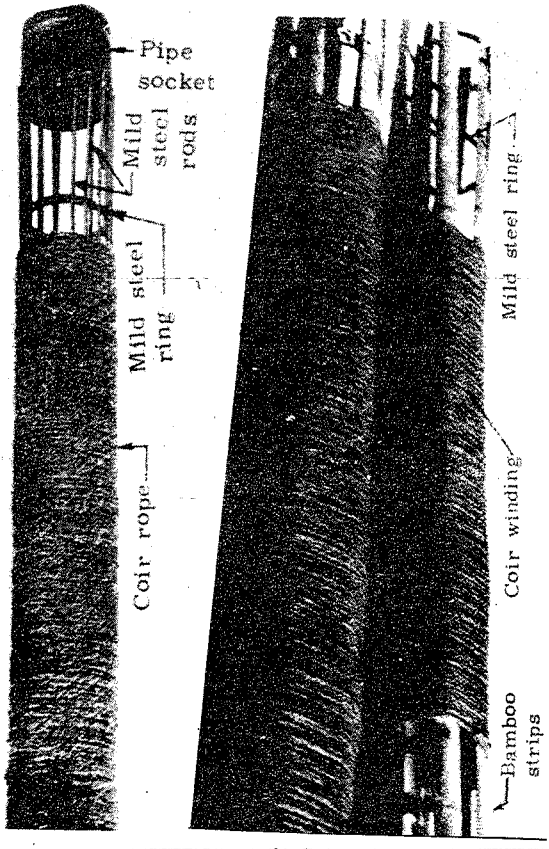
In the above design, since the wire netting is not in direct contact with the perforated pipe over which it is wrapped, the area of perforations is not decreased by the netting in front of the perforations. The arrangement therefore provides good hydraulic efficiency. Such a strainer could last for 10 years or so.

The biggest disadvantage of such a strainer however, is that it involves bimetallic construction, since the pipe base is of steel and the outer mesh jacket is of brass or bronze, etc. Such bimetallic construction causes electrolytic action and corrosion of the steel pipe. To overcome this difficulty, several designs by using the same type of metal in the base pipe as well as in the net jacket have been commercially produced, but are generally very costly, since single metal construction in brass or copper considerably increases the cost of the strainer. Such costly single metal strainers are however, widely used in developed countries and are sold in several brand names.

(*iv*) **Coir rope strainers** are low cost strainers, generally used for shallow irrigation tubewells, particularly in deltaic regions of developing countries. Such a strainer essentially consists of a cylindrical frame made of iron rods or bamboo strips, wound round by coir rope of 3 to 5 mm diameter, as shown in Fig. 16.51.

The disadvantage of such a strainer is its short life of 3 to 5 years. The cause of its failure is the rusting of iron bars of the supporting frame, and the loosening of the coir rope which expands on wetting. Use of *nylone rope*, and coating the iron rods with bitumen have been found to increase its life to a good extent. Since the coir rope strainers cannot withstand the high pressure created during the development of the tubewell by an air compressor, wells using such screens shall have to be developed only by pumps.

Closely mounted *polythene rings* of 6 to 10 mm width have also been used in place of coir rope to make such a stainer to be more resistant to corrosion and deterioration.



(*a*) Coir rope strainer with mild steel frame (*b*) Coir rope stainer with bamboo frame.

Fig. 16.51. Coir rope strainers

16.25.6. Construction and Boring of Tubewells. With respect to the method of boring adopted, the tubewells can be classified into the following three categories;

(1) *driven tubewells;*
(2) *Jetted tubewells;* and
(3) *drilled tubewells.*

These three types of borings are discussed below :

16.25.6.1. Driven Tubewells. A driven tubewell consists of a pipe and a well point fixed to the lower end of the pipe, as shown in Fig 16.52. The assembly is forced down the ground to penetrate into the water bearing formation, by driving it with a wooden maul, drop hammer, or other suitable means.

If the water level is 7 m or more below the ground surface, the well pipe should be more than 5 cm in diameter, so that a jet or a cylinder may be inserted and submerged to permit the pump to function.

A special device, called a *cap* or a *drive head* is provided at the top of the pipe assembly during the driving operation, so as to protect the pipe during hammering. After

each length of the pipe is hammered into the ground, the cap is removed and additional sections are attached, and driven as required.

The pipe is kept full of water at all times. In its descent through the aquicludes, the water in the pipe shall not flow out through the screen, since the screen is sealed by the impervious formation; but as the well point reaches the water bearing formation, some water will flow out of the pipe, and water remaining in the pipe drops to a static level. This is a signal to the driller that water bearing formation has been reached.

To develope the well, the driller attaches a small pitcher pump to the pipe. A considerable amount of the sandy water may be pumped for a short time, but if a good aquifer has been tapped, continued pumping should result in clear sand free water.

Driven tubewells yield very small discharges, and are suitable only for individual domestic supplies. Their construction is limited to shallow depths in soft unconsolidated formations free from boulders, and other obstructions.

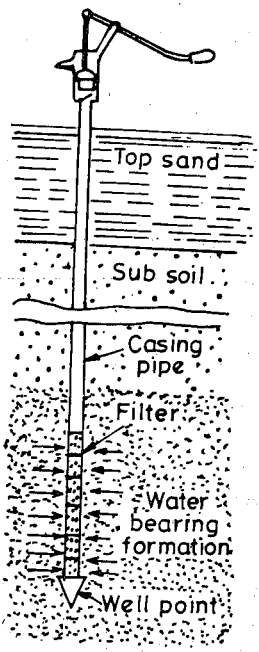


Fig. 16.52. Driven tubewell

16.25.6.2. Jetted tubewells. A jetted tubewell may be constructed either with a hand operated equipment or power driven machines, depending upon the soil formation and size and depth of the well bore.

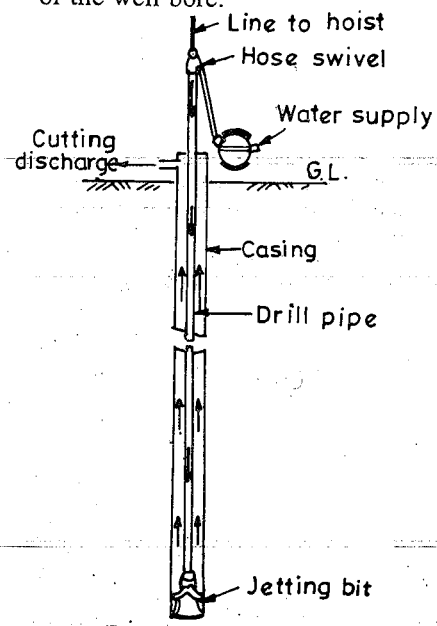


Fig. 16.53. Jetting method of drilling shallow tubewells.

A hole in the ground is made by the cutting action of a stream of water, which is pumped into the well through a small dia pipe, and forced against the bottom of the hole through the nozzle of a jetting bit (Fig 16.53). A casing pipe is provided during boring to prevent the caving of the bore/hole.

After the hole has been jetted down to the designed depth, the well assembly consisting of blank pipes and screen, is lowered, and *the outer casing is pulled out.*

Jetted tubewells have small yields, and their construction is feasible only in unconsolidated formations.

16.25.6.3. Drilled tubewells. Deep and high capacity wells are constructed by drilling. Various techniques are employed in drilling the well hole. Different techniques have comparative advantages and disadvantages over each other, depending upon the type of formation to be drilled. Therefore, each well should be treated as an individual project, and one particular method adopted depending upon its suitability. Some of the **drilling methods** commonly used, are described below :

(1) **Standard method or Cable tool method of drilling.** This method of drilling the well hole is known as *percussion drilling*; because in this method, the

well hole is made by percussion (*i.e.* by hammering and cutting). This method is very useful for cutting consolidated rocks from soft clay to hardest rocks, and is generally unsuitable in loose formations, such as unconsolidated sand and gravel or quick sand. This method becomes ineffective in loose material, because the loose material slumps and caves around the *drilling bit.* The drill bit has a chiesel sharp edge which breaks the rock by impact when alternately lifted and dropped. This drilling bit is connected at the lowest end of the entire 'falling and rising arrangement' known as the *string of tools.* [Refer Fig. 16.54 (*a*) and (*b*)]. From top to bottom, the string of tools consists of a rope socket, a set of jars, a drill stem, and the drilling bit.

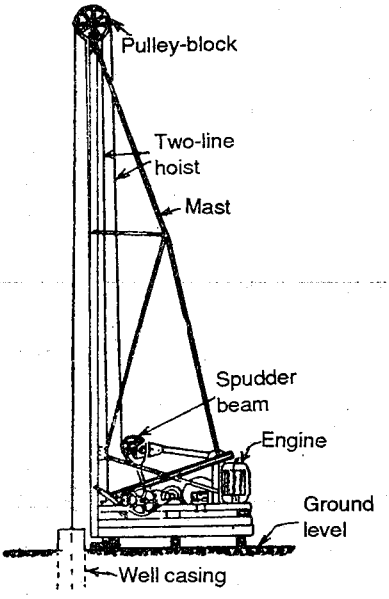


Fig. 16.54 (*a*) Basic components of Percussion Drilling Rig.

Tools are made of steel and are joined with tapered box and pin screw joints. The entire assembly weighs several tonnes. The most important tool of the entire assembly is the drilling bit (or drill) as it does the actual rock cutting. The drill stem is the long steel bar which adds weight and length to the drill, so that it can cut rapidly and vertically.

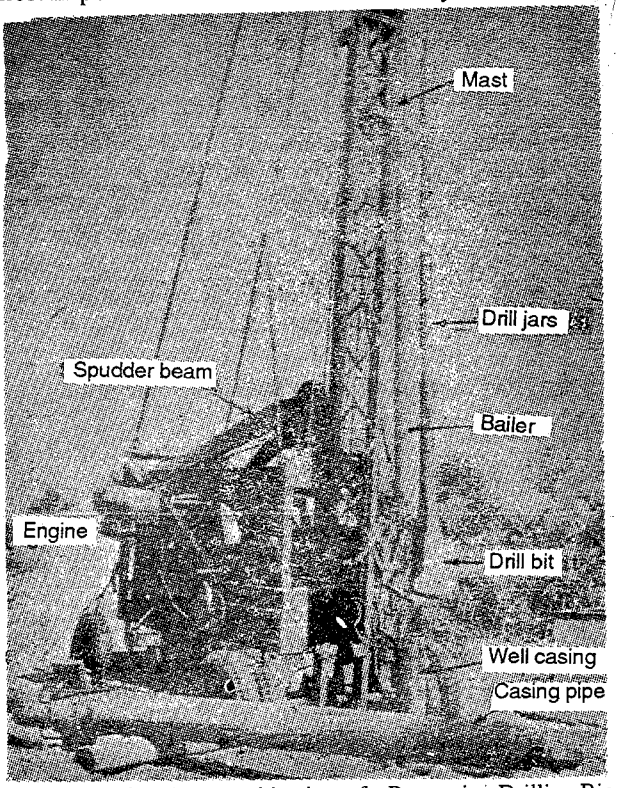


Fig. 16.54 (*b*). Photographic view of a Percussion Drilling Rig. The string of tools is seen to the right of the mast.

The set of jars have no direct effect on the drilling. They only loosen the tools when they stick in the hole. A rope or a cable is fastened at the upper end to the rope socket and to a dead man (or a heavy weight) at the lower end.

The entire assembly of tools is suspended from an assembly of a mast and walking beam, etc. This assembly, known as *drilling rig*, in turn, is generally mounted on a truck, as to make it easily portable. The mast should be sufficiently high so as to allow the longest of tools to be hoisted.

As the drilling proceeds, the tools makes 40 to 60 strokes per minute, from a height of 0.4 to 1 m. Water is sometimes added in the hole so as to form a paste with the cuttings, thus reducing friction on the falling bit. After the bit has cut 1 to 2 m through a formation, the string of tools is lifted out and the hole is cleaned and cleared of the cuttings by means of a *bailer*. The process is known as bailing out the hole.

A bailer essentially consists of a pipe with a valve at the bottom and a ring at the top. When lowered into the well, the valve permits the cuttings to enter the bailer but prevents them from escaping the bailer. After it is filled with cuttings, it is lifted up to the surface and emptied.

In unconsolidated formations, casing should be driven down and maintained near the bottom of the hole to avoid caving. Casing is driven down by means of drive clamps fastened to the drill stem. The up and down motion of the tools, striking the top of the casing, protected by a drive head, sinks the casing. On the bottom of the casing, a drive shoe is fastened to protect the casing, as it is being driven.

(2) **Hydraulic rotary or Direct rotary method of drilling.** This is the fastest method of drilling and is especially useful in unconsolidated formations. The method involves a continuously rotating hollow bit, through which, a mixture of clay and water or mud is forced. The bit cuttings are carried up in the hole by the rising mud. No casing is required during drilling because the mud itself makes a lining on the walls of the hole which prevents caving.

The drill bit is connected to a hollow steel rod (or drill stem), which, in turn, is connected at the top to a square rod, known as the Kelley [Refer Fig. 16.55 (*a*)]. The drill is rotated by a rotating table which fits closely around the Kelley and allows the drill rod to slide down, as the hole progresses.

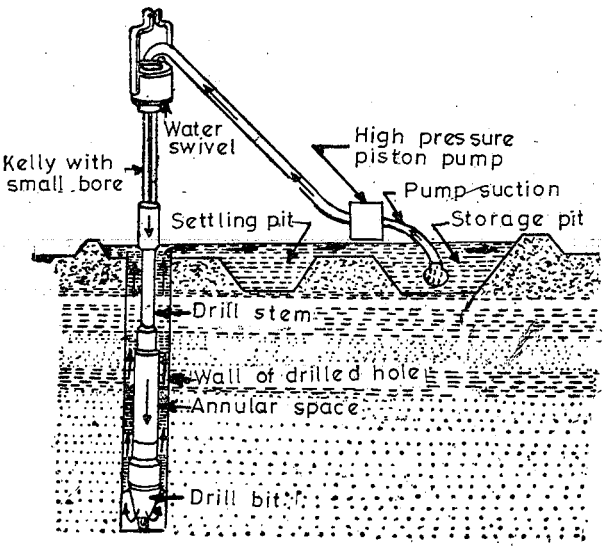


Fig. 16.55 (a). Schematic sketch illustrating the basic principles of Direct Rotary Drilling.

The drilling rig, such as shown in Fig. 16.55 (*b*), consists of a mast, rotating table, a pump for forcing the mud, a hoist and the engine. The mud, after it emerges out of the hole, is carried to a tank where the cuttings settle out and the mud can be repumped into the hole.

After the drilling is completed, the casing is lowered into the hole. The clay deposited in the well-walls during mud pumping, is removed by washing it with water. Water containing some chemicals like sodium hexametaphophate is forced through the drill rod and the washings come out through the preforations of the casing. When the washing at one level is completed, the bit is raised, and the process repeated.

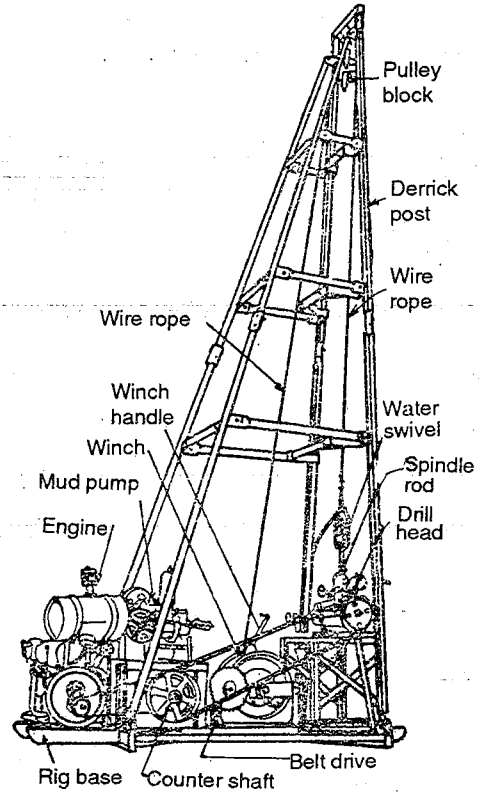


Fig. 16.55 (*b*). Skid mounted light duty Rotary drilling rig.

(3) **Reverse rotary method of drilling.** A modification of the hydraulic rotary method is known as the reverse rotary method. This is gaining popularity day by day. *It is quite useful for making large wells (diameter up to 1.2 m. app.) in unconsolidated formations.*

The tools consists of a hollow drill, a drill pipe and water swivel. In this method, the cuttings are removed by water through a suction pipe called the *drill pipe*. The equipment consists of a mast or a derrick, a centrifugal pump, necessary water and power arrangement, and the requisite casing pipe.

The hole is driven by pumping water under pressure through the drill bit, while it is churned up and down. The walls of the hole are supported by hydrostatic pressure acting against a film of fine grained material deposited on the walls by the drilling water. Cuttings are removed by the

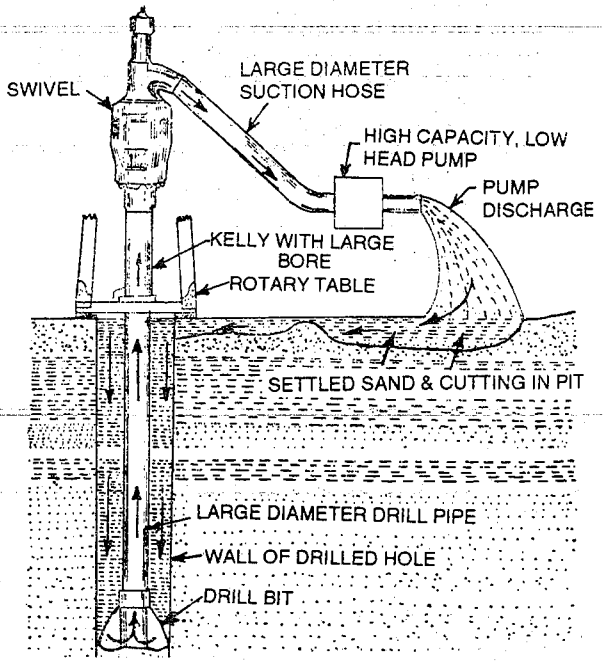


Fig. 16.56. Schematic sketch illustrating the basic principle of Reverse Rotary Driling.

water, and after the mixture (water + cuttings) comes out to the surface, it is passed through a settling tank (Refer Fig 16.56)

The sand settles out here but the fine grained particles are recirculated, so as to help in stablising the walls. Casing and cleaning of the walls, etc. is the same as in the hydraulic rotary method.

Comparison of cable tool and hydraulic rotary methods of drilling. *Advantages of Cable Tool Method are given below:*

1. A more accurate sample of the formation can be obtained.
2. Lesser amount of water is required during operations.
3. Cable tool rig is lighter and easy to transport.
4. Very useful for consolidated rocks and less useful for loose formations.
5. For shallow tubewells in unconsolidated materials too, it comes out to be cheaper.

Advantages of Hydraulic Rotary Method are given below :

1. Can be used for larger holes up to 1.5 m diameter.
2. Can be best used for drilling test holes, because the hole can be abandoned with minimum cost.
3. Rotary drilled hole can be gravel packed, which increases its specific capacity, and keeps the fine particles away, thus causing less sand trouble.
4. Casing is to be driven only after the hole completion, and hence, can be set at any desired depth.
5. It is the fastest method of drilling and especially useful in unconsolidated formations.
6. It can handle alternate hard and soft formations with ease and the danger of accidents is lesser. In quick sands, clays, etc., cable tool method is likely to give troubles, as there is a danger of freezing.

Verticality of the bore hole during drilling must by ensured, irrespective of the method adopted for drilling the bore hole. A common specification allows a deviation of 15 cm from the vertical in a length of 30m.

16.25.6.4. Well Log.

During drilling a well hole, the description of the material encountered in sequence through the drilling, has to be recorded, and this record is called a well log. A well log will, therefore, record the different types of formations with their correct depths of occurrences. Samples of drill cuttings at different depths, at regular intervals of 1.5 to 2 m, are also collected, to determine the exact nature of the rocks being drilled.

The prepared well log will help in designing and commissioning the tubewell :

16.25.7. Installing Well Screens. Well casings and well screens are installed on the basis of the well log. If the strata conditions, as revealed by the well log, warrant a cavity type tubewell, no screen will be necessary.

In such a cavity well therefore, the casing pipe used in the drilling, may either be left as it is, or if it is costly, then a smaller dia blind well pipe may be inserted, and the casing pipe withdrawn.

In screen tubewells, strainers or slotted pipes will be required, and the procedures for installing such well strainers shall vary with the design of the well and the method employed for drilling. The screens or slotted pipes shall have to be located opposite the water bearing strata, in order to draw water from the strata. This is done by assembling together the whole length of the screens and the blank pipes in exactly the same order and the lengths, in which they are to be lowered in the bore.

The screens and the blind pipes are lowered into the bore one by one, starting from the bottom end. During lowering of these well pipes, the casing pipe may be withdrawn, to allow the screens and the blind pipes to hold them in position.

In unconsolidated formations, sometimes, the casings are left intact, to support the bore hole and to freely admit water into the well. In such a case, the casing pipe itself should either contain *perforations* or its lower part be replaced by a screen or a strainer.

16.25.8. Well Development. Tubewells are developed to increase their specific capacity, prevent discharge of sand, and to obtain maximum economic well life. Development means *the stabilisation of the walls of a well adjacent to the screen, by a process which removes the fine particles from the formation immediately surrounding the well screen, leaving coarser particles to contact and surround the screen.*

The *main objectives of well development* are:

(*i*) to unclog the water bearing formation

(*ii*) to increase the porosity and permeability of the water bearing formation in the vicinity of the well.

(*iii*) to stabilise the sand formation around a screened well, so that the well may yield sand free water.

Development is necessary in all gravel packed wells and other screened wells, except when the screen is made of fine wire mesh or coir or other closely knit filters, located in a highly permeable aquifer.

The basic principle in well development is to cause reversals of flow through the screen openings, that will rearrange the aquifer particles. This is essential to break down *bridging* of the groups of particles. Fig 16.57 shows how small particles can bridge between large particles across the screen openings, when the flow of water is in one direction only. Reversing the direction of flow by *surging the well* does help in removing the bridging.

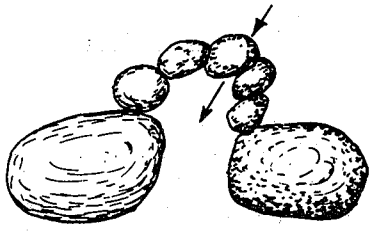


Fig. 16.57. Bridging action caused by one directional flow.

The commonly adopted **well development methods** are discussed below :

(*i*) **Well development by surging.** In this method, a plunger is worked up and down in the well, so that the water is alternately forced out into the surrounding formation and then allowed to flow back into the well. A *surge block* or *surge plunger* is the tool, which is used for this purpose.

The outflow portion of the surge cycle breaks down the bridging to loosen the fines; and the inflow portion of the surge cycle moves the fines towards the screen and into the well, from where they are removed.

(*ii*) **Well development by pumping.** In this method, a tubewell may be developed either by *over pumping* or by *rewhiding the well.*

Overpumping involves heavy pumping of the well to cause heavy drawdown. This is not a very effective method, for well development because the flow of water remains unidirectional, thus not removing the bridging of the particles.

Rewhiding involves starting and stopping of pumping intermittently to provide relatively rapid changes in the head of the well. While this may be done with any type of pump, it is most effectively done with a turbine type of pump installed with a foot valve.

(*iii*) **Well development by compressed air.** This method is most commonly adopted for developing wells, and may involve either :

(*a*) *backwashing technique,* or (*b*) *surging technique.*

A combination of both the techniques may sometimes be used for more effective development.

Backwashing technique involves forcing of the well water back into the aquifer by means of compressed air introduced into the well through the top of the casing after it has been closed. When the pressure is released, the water will flow back into the well through the screen to bring the fine particles from the area surrounding the well, thus ensuring their removal. The process is continued till no sand is brought in.

Surging technique involves the principles of both the pumping method and the surging method described earlier. The inrush of compressed air creates a powerful surge within the well, and loosens the fine material surrounding the perforations, which may then be brought into the well by continuous air injection. The operation is repeated at intervals along the screened section of the well, until sand arrival is stopped. The principle of pumping is the same as described eariler.

(*iv*) **Well development by jetting.** Jetting with water at high velocity is an effective method of well development. The method involves operating a horizontal water jet inside the well in such a way that the high velocity water stream shoots out through the screen openings. Fine particles are thus washed out of the aquifer, and the turbulence created by the jet brings these fines back into the well through the screen openings above and below the point of operation. By slowly rotating the jetting tool, and by gradually raising and lowering it, the entire surface of screen can be covered.

Use of dispersing agents in well development. Certain chemicals like tetra sodium pyrophosphate, sodium tripolyphosphate, sodium hexametaphosphate, and sodium septaphosphate, etc., when added to the well water, and to the water used in *backwashing* or *jetting techniques* of the well development described above, considerably helps in mud removal, thereby increasing the effectiveness of the above methods.

16.25.9. Spacing of Tubewells. Tubewells have been generally located without any scientific studies, till recently. It was thought that for Indo-Gangetic area, tubewells yielding about 45 litres/sec could be located within 2.4 sq. km., which fixed the cultivable commanded area of the tubewell to about 400 hectares. With further research now, tubewells are constructed with a command of 120 hectares, and on an average, a tubewell is located in 1.5 sq. km, and the resultant large scale pumping has not materially lowered the watertable.

16.25.10. Life of a Tubewell and Reasons for its Failure. A normal tubewell lasts for about 15 to 20 years in Northern India. It may fail due to (*a*) Incrustation, or due to (*b*) corrosion.

(*a*) **Incrustation.** The incrustation of the well pipe occurs due to the deposition of alkali salts on the inside walls of the pipe. The important salt causing incrustation is *calcium carbonate,* although calcium and magnesium sulphates and silicates may sometimes be the basic binders. The incrustation of the well pipe reduces the effective diameter of the well pipe, and hence reduces the discharge of the tubewell.

The incrustation can be reduced :

(*i*) by reducing the drawdown and hence the pumping rate ;

(*ii*) by using screens having larger area of openings (or larger diameter pipes) so as to allow some allowance for the future incrustation ;

(*iii*) by using such materials for the strainers (screens) that may easily permit the removal of incrustating material by chemical action without affecting that strainer material. In other words, the acids, etc., which are used to remove the incrustation, should not produce any effect on the strainer materials ; and

(*iv*) by properly maintaining and periodically cleaning the well screens.

The incrustation can be delayed or reduced by these four methods, although it cannot be completely eliminated and thus the life of the tubewell can be increased by the above methods.

(*b*) **Corrosion.** The well pipe is gradually destroyed by corrosion due to the action of acidic water on the pipe material. When chlorides and sulphates or carbon dioxide are present in the water, the well pipe will definitely get corroded. The aquifer sand, surrounding the well pipe, finds a way out into the corroded pipe through the worn out pipe walls, thus bringing sand along with water. Hence, *corrosion results in excessive withdrawal of sand along with well water.*

Thicker pipes may be used to avoid corrosion. Stainless steel strainers will be most suitable, but are very costly. In affluent countries like U.S.A., such stainless steel screens are being progressively used, but are too expensive for the developing countries like India. An iron or steel screen can be depended upon only for a limited service life of the tubewell in most of the waters. The life of such a screen can be increased by galvanising (*i.e.*, by zinc coating) the pipe material. Other measures, which can help in reducing corrosion, and thereby increasing the life of the tubewell, are :

(*i*) by reducing the drawdown and the pumping rate ;

(*ii*) by reducing the flow velocity by increasing the percentage of the open area or the diameter of the well pipe ;

(*iii*) by using thicker pipes ;

(*iv*) by using corrosion resistant materials for the pipes ; and

(*v*) by using corrosion resistant coatings on the pipes.

16.25.11. Design of a Strainer Tubewell. The design of a strainer type tubewell essentially consists of designing the size of the tube, the size of the bore hole, the length of the strainer, and type and horse power of the pumping arrangement required to lift the water. The design considerations for these parts of a tubewell are discussed below:

(*i*) **Size of the tube.** The diameter of the tube (pipe) is decided from the considerations of permissible flow velocity through the tube. Since the strainers are installed at different levels, the velocity of water in a tube of a fixed size will not be constant but will be increasing towards the top. Hence, it is theoretically possible to reduce the size of the tube from the top towards the bottom, such that the velocity is more or less constant throughout the tube length. However, it is generally not economical to install varying size tube, and hence, a single sized or at the most a two sized tube (in case the aquifers are available at too much different depths) may be used in actual practice. The velocity through the tube may be limited between 1.5 to 4.5 m/sec. Knowing the design discharge of the tubewell and the velocity, the area of the tube and hence, its diameter can be easily calculated. The nearest available size in the market may then be used.

(*ii*) **Size of the bore hole.** Normally, the size of the bore hole should be at least 5 cm bigger than the size of the tube, so as to facilitate the lowering of the tube in the hole. If gravel packing is provided then the bore size will be decided by making provision for the gravel thickness also.

(*iii*) **Length of the strainer**. After deciding the diameter of the tube, the length of the strainer required to obtain the design discharge may be calculated for unconfined or confined aquifer cases, respectively, as given below :

(*a*) *For Unconfined Aquifer Case*; from eqn. (16.17) we have

$$Q = \frac{\pi \cdot K \cdot (d^2 - h_w^2)}{2.3 \log_{10} R/r_w}$$

If s is the drawdown or depression head, then w.r.t. to Fig. 16.13, we can easily write

$$d - h_w = s$$

Now,
$$Q = \frac{\pi \,.\, K\,(d - h_w)\,(d + h_w)}{2.3 \log_{10} \frac{R}{r_w}} = \frac{\pi\, K.s.\,(\overline{h_w + s} + h_w)}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$Q = \frac{2\pi K.s.\left(h_w + \frac{s}{2}\right)}{2.3 \log_{10} \frac{R}{r_w}} \qquad ...(16.17\ a)$$

or
$$\left(h_w + \frac{s}{2}\right) = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

or
$$h_w = \left[\frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s} - \frac{s}{2}\right] \qquad ...(16.17\ b)$$

h_w in this case, represents nothing but the **length of the strainer** needed. In the above equation, r_w is the radius of the well and is known by now. The value of the radius of influence (R) may be assumed between 250 to 500 m. Such a wide variation in the value of R will, at the most change the discharge up to a maximum of 12%, since the relation between Q and R is logarithmic. The suitable value of permeability coefficient (K) may be assumed. The design discharge and the depression head (*i.e.* drawdown) are also known; hence, the value of h_w *i.e.*, the length of the strainer can be easily calculated.

(*b*) *For Confined Aquifer Case*; from eqn. (16.20) we have

$$Q = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$H = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s}$$

In this equation, H represents the length of the strainer which can be easily calculated. This designed strainer length is provided in one or more aquifers, depending upon the site availabilities.

(*iv*) **Type of pumping arrangement.** Three types of pumps are generally used in order to lift water. They are

(1) *Centrifugal pumps*
(2) *Bore hole type pumps*, and
(3) *Jet pumps*.

These three types of pumps are discussed below :

(1) *Centrifugal Pumps.* A centrifugal pump lifts water from the lower level to a higher level by creating the required pressure with the help of centrifugal action. The

maximum suction head, under which the pump can practically work effectively, is about 6 to 8 m. Hence, such a pump can be used only at places, where the fluctuations in watertable plus the depression head is limited to a maximum of 8 m, as shown in Fig. 16.58. For larger values, the bore hole type pump is to be used. A centrifugal pump is generally available in two models, *i.e.* (*i*) when the pump and the motor are built together, and (*ii*) when the pump and motor are separately built and coupled together. Such centrifugal pumps are polularly known as **monoblock** centrifugal pumps.

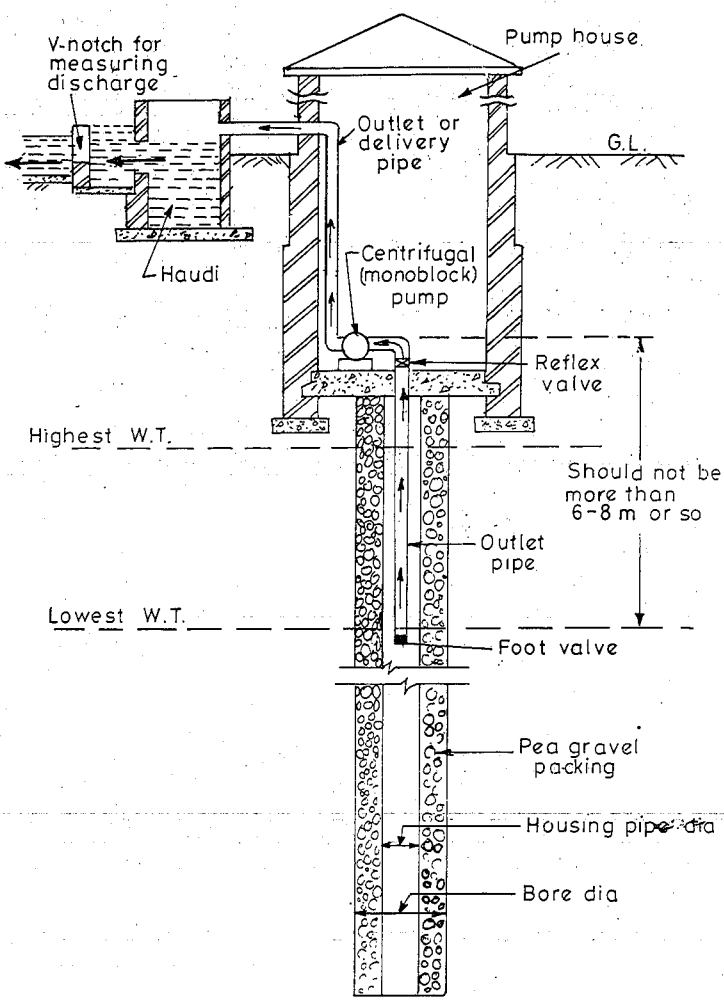


Fig. 16.58. Section of a tubewell using Monoblock Centrifugal pump.

A section of tubewell using a centrifugal pump is shown in Fig. 16.58. In this arrangement, a sump well is sometimes constructed, so as to place the pump at a required lower level, below the ground level. The pump is to be placed slightly above the highest water table, so as to avoid its submergence. The minimum water level should not be lower than the pump level by more than 6 to 8 m as shown, otherwise the pump will not work.

Hence, such an arrangement can be used only where the water is available at smaller depths from the surface, (*i.e.* generally for shallow cavity wells) and it cannot be used for those *deep tubewells* where water is generally available quite deep.

The sump well is also generally of a large size and deep enough, and has to be plugged at the bottom with concrete. This arrangement may sometimes make it costly, although this type of pump is the cheapest. Hence, such type of pumping arrangement is genrally not preferred in modern days, when the bore hole type pumps are being increasingly used.

(2) *Bore hole type Pumps.* Such pumps consist of special centrifugal pump impellers connected in series, mounted on a vertical shaft, and driven by a motor. They are of small diameter and can be lowered in the casing pipe itself. The top 20 to 30m of the bore hole and the casing pipe is generally kept wider than the remaining normal bore, so as to accommodate the pump bowl in the casing pipe.

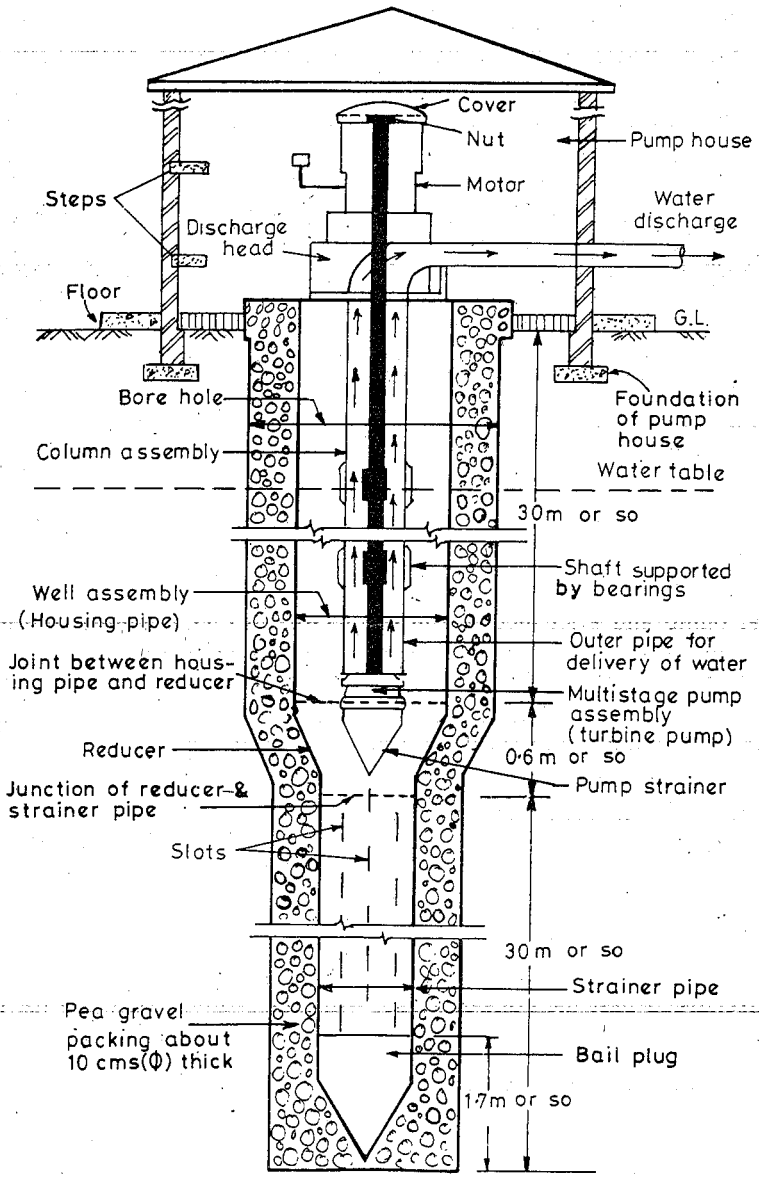


Fig. 16.59. Section of a tubewell using a turbine type of a bore hole pump.

Two types of bore hole pumps, *i.e. (i) submersible pumps* ; and *(ii) turbine type of pumps*, are available. In **submersible pumps,** the motor and the pump are both attached together and lowered inside the bore; whereas in a **turbine type**, the pump is driven by a direct coupled electric motor of a vertical shaft type, and, is placed at the top of the line shaft at the ground level. The necessity of constructing a sump well is thus completely avoided, which may make this arrangement a cheaper and a better alternative to the monoblock centrifugal pump, although the cost of such a pump is higher, and lowering by chain and pulley is difficult. Even among these two types of available pumps, submersible pump costs less than that of a turbine type. A section of a tubewell using turbine pump installation is shown in Fig. 16.59. A submersible pump is also shown in Fig. 16.60.

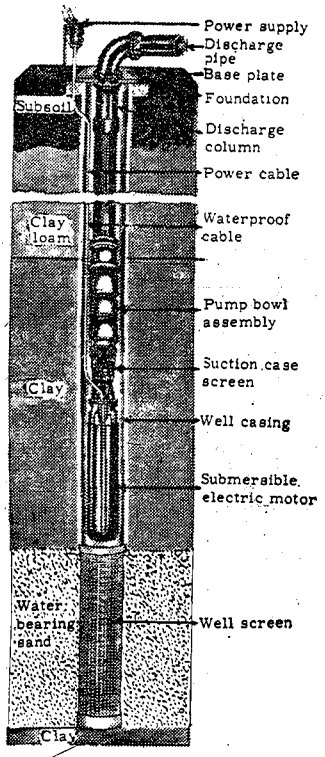
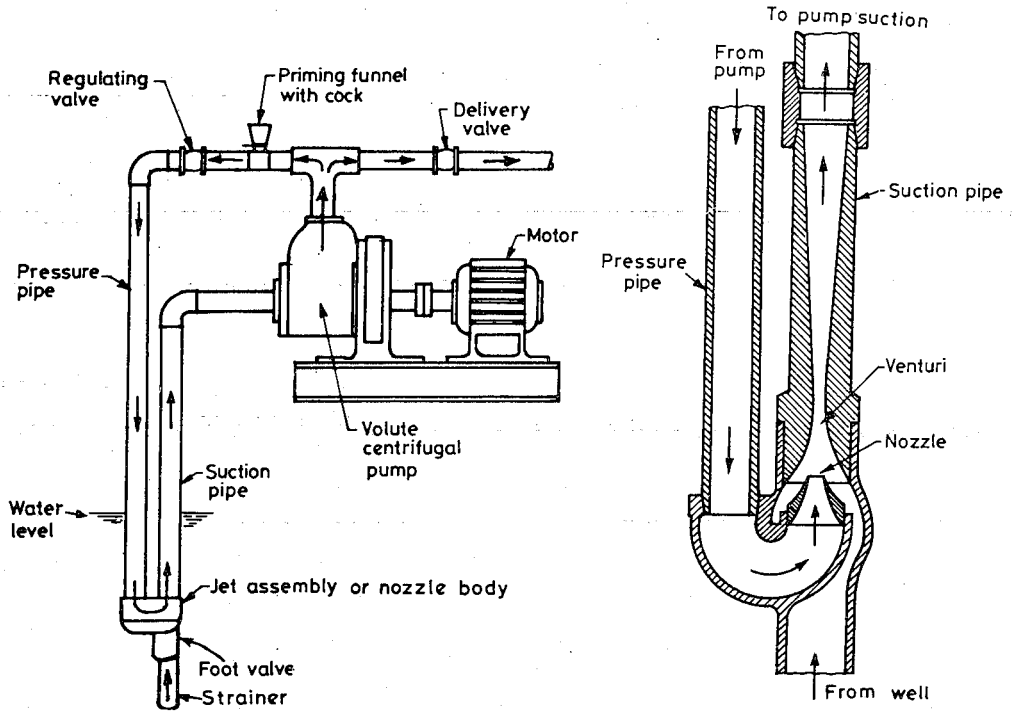


Fig. 16.60. A submersible pump.

(3) *Jet Pumps*. Jet pumps are also sometimes used these days for lifting water from the smaller tubewells installed for individual domestic supplies, where the water is not available within 8 m suction lift, thereby not permitting the use of the monoblock centrifugal pumps. Jet pumps can be used for suction lift of 6 to 30 m or so. A **jet pump** consists of a combination of a centrifugal pump and a jet mechanism or ejector. In this assembly, the motor and the pump constitute a small unit like a centrifugal monoblock pump, and is placed at the ground level. A nozzle and a venturi assembly connected with the pressure pipe is joined with the main well pipe, as shown in Fig. 16.58. The pressure pipe is connected on the outlet side of the pump along with the delivery pipe. A portion of the high pressure water thus returns through the pressure pipe to activate the nozzle in the ejector. The nozzle is shaped so that it smartly and abruptly reduces the area through which the flow must pass, thereby increasing the velocity of flow. This creates a pressure area around the venturi, which draws more water from the well. The vacuum created by the impeller of the pump placed at the ground surface, draws the flow through the suction pipe. The water is pumped out through the delivery pipe at the desired pressure. The additional supply of water which is obtained from the well is discharged past the control valve, while the volume required for producing the flow is recirculated through the pressure pipe. The control valve is set to maintain the necessary pressure to produce the flow at the existing pump head.

Jet pumps cannot be started until the whole system is filled with water. No priming is however required with the installation of foot valve, as in a monoblock pump. The discharge of the jet pumps is about 20 litres per minute against the head of 15–30 m above the ground level.

Jet pumps are, however, generally not used for irrigation tubewells, because of their low efficiency, which generally is of the order of 35% as compared to 66–85% for the



(*a*). Jet pump installation.

(*b*). Sectional view of the jet assembly of the jet pump.

Fig. 16.61

other types of pumps. Moreover, jet pumps are not easily adapted to the locations where ground water levels are subject to large seasonal fluctuations, or where severe corrosion or incrustation may cause enlargement or blocking of the jet nozzles.

(*v*) **Horse power of the pump motor**

The power of the motor $= \dfrac{\gamma_w \cdot QH}{\eta}$ Nm/s (*i.e.* watts)

$$\text{H.P. (metric)} = \frac{\text{watts}}{735} = \frac{\gamma_w \cdot QH}{735\,\eta} \qquad \ldots(16.47)$$

where Q = Discharge to be delivered in cumecs

H = Total lift, *i.e.* the head against which the motor has to work, in metres

γ_w = Unit weight of water in N/m^3

η = Efficiency of the pump set, and is generally taken as 65% (*i.e.* 0.65) for centrifugal or bore hole pumps.

The total head (H) against which the motor has to work consists of :

(a) Maximum depth of water level below the ground level.

(b) Maximum depression head.

(c) Velocity head.

(d) Frictional losses in the tube pipe, including the losses at bends and strainer openings.

The frictional loss in the pipe (h_f) may be calculated by using the equation

$$h_f = \frac{f' \cdot l \cdot V^2}{2gd} \qquad ...(16.48)$$

where f' = Coefficient of pipe friction which is generally taken to be 0.024

l = Total length of the pipe line, including the vertical as well as horizontal lengths

V = Velocity of flow in the pipe

d = Diameter of the pipe line.

The losses at the bends and those at the entry of the strainer openings are generally taken to be equal to 25 to 30% of the frictional losses in the pipe.

Knowing the total value of H, *i.e.* the head against which the motor has to work ; the horse power of the motor can be easily calculated.

Example 16.14. *Design a tubewell to deliver 33,000 gallons per hour at a depression head of 5 m. The average water level is 10 m below the ground in October and 15 m in July. The geological investigation has yielded the following results at the site of boring :*

Depth	*Type of Strata*
0 to 5 m	*Surface clay*
5 to 20 m	*Very fine sand*
20 to 30 m	*Clay with Kankar*
30 to 50 m	*Coarse sand*
50 to 60 m	*Clay*
60 to 70 m	*Medium sand*
Below 70 m	*Clay with sand stone.*

Solution.

(i) **Design of the well pipe**

Assume the velocity of flow in the tube = 2.5 m/sec

Discharge required = 33,000 gallons per hour = $33{,}000 \times 4.55$ litres per hour

$$= \frac{33{,}000 \times 4.55}{1{,}000} \text{ cubic metres per hour}$$

$$= \frac{33 \times 4.55}{60 \times 60} \text{ cubic metres per sec. (}i.e.\text{ cumecs)} = \mathbf{4.17 \times 10^{-2}\ cumecs}$$

Area of the tube or well pipe

$$= \frac{\text{Discharge}}{\text{Velocity}} = \frac{4.17 \times 10^{-2}}{2.5} \text{ m}^2 = 167 \text{ cm}^2$$

Diameter of tube $(d_w) = \sqrt{\frac{4}{\pi} \cdot A} = \sqrt{\frac{4}{\pi} \times 167} = 14.6 \text{ cm}$

Use 15 cm dia pipe. The actual velocity of flow in the pipe of 15 cm dia

$$= \frac{4.17 \times 10^{-2}}{\frac{\pi}{4}(0.15)^2} = \mathbf{2.36 \text{ m/sec}}$$

(*ii*) **Size of the bore hole.** The size of the bore hole should be 5 cm more than that of the pipe. Hence, use 20 cm dia bore hole.

(*iii*) **Length of the strainer**. From the geological investigation report, it is evident that the main aquifers are confined between clay strata. Hence, the discharge formula for confined stratum will be used to work out the strainer length. Using equation (16.20), we have

$$Q = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

or
$$H = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi K.s.}$$

where $Q = 4.17 \times 10^{-2}$ cumecs

r_w = Radius of the tube $= \frac{0.15}{2} = 0.075$ m

s = Depression head = 5 m (given)

R = 350 m (assumed)

K = Permeability coefficient for sand

$= 0.04$ cm/sec $= 0.04 \times 10^{-2}$ m/sec

(assumed — Table 16.2)

$$\therefore \quad H = \frac{2.3 \times 4.17 \times 10^{-2} \log_{10} \frac{350}{0.075}}{2 \times 3.14 \times 0.04 \times 10^{-2} \times 5}$$

$$= \frac{2.3 \times 4.17 \times 3.669}{2 \times 3.14 \times 0.04 \times 5} = 28.01 \text{ m; say } 28 \text{ m}$$

Hence, use the length of the strainers = **28 m.**

Check for the entrance velocity (v_e)

Using eqn. (16.46) : $h = \frac{Q}{A_0 \cdot v_e}$, we get

$$28 = \frac{4.17 \times 10^{-2}}{(\pi \times 0.15 \times 15\%) \times (v_e)}$$

(assuming 15% opening area in the screens)

or $$v_e = 0.0197 \text{ m/s} = 1.97 \text{ cm/s} < 2 \text{ cm/s (O.K.)}$$

Note : If v_e is found to be more then the safe entrance velocity of 2 to 3 cm/s, then either the dia of the screen or the length of the screen shall have to be increased. In case this is not practically feasible, then the percentage of opening area shall have to be increased from 15% to about 20 to 25% (preferably 20%).

The selected 15 cm dia screen in 19 m length can now be provided in the coarse sand aquifer (lying at depth between 30 to 50 m), and 9 m length can be provided in the medium sand aquifer (lying at depth between 60 to 70 m), so as to provide a total 28 m screen length. In both the cases, 0.5 m aquifer depths shall be left unscreened at either ends.

Note : This arrangement will lead to the screening of 95% of the aquifer depth,as against the preferred value of 90%. Even this anomaly can be corrected by using a screen of 20 cm dia, and recalculating the length of the screen.

(*iv*) **Type of pump required.** Since the fluctuations of watertable (*i.e.* 15 m– 10 m= 5 m) plus the depression head (5 m) total to be 10 m, which is more than 6 to 8, centrifugal pump is out of question. Bore hole type of pump shall, therefore, be used.

(*v*) **Horse Power of the motor**

$$\text{H.P.} = \frac{\gamma_w \cdot QH}{735\eta} \quad \text{in S.I. units}$$

where $Q = 4.17 \times 10^{-2} \text{ m}^3/\text{s}$

$\eta = 0.65$

$\gamma_w = 9.81 \text{ kN/m}^3 = 9.81 \times 10^3 \text{ N/m}^3$

H = Total head against which the motor has to work

= Maximum depth of watertable + Depression head + Velocity head + Losses.

where, (*a*) Maximum depth of watertable = 15 m (given)

(*b*) Maximum depression head= 5 m(given)

(*c*) Velocity dead

$$= \frac{V^2}{2g} = \frac{(2.36)^2}{2 \times 9.81} = 0.29 \text{ m}$$

(*d*) Loss of head due to friction in pipe (h_f) plus losses at bends in strainer, etc. (say 0.25 h_f) where $h_f = \dfrac{f' \cdot l \cdot V^2}{2gd}$

where, $f' = 0.024$

l = Length of pipe line

= Vertical length of pipe including the blind pipe and strainer + Horizontal length of delivery pipe

$$= 70 \text{ m} + 10 \text{ m} = 80 \text{ m}$$

$$\therefore h_f = \frac{0.024 \times \times 80 \times (2.36)^2}{2 \times 9.81 \times 0.15} = 3.64 \text{ m}$$

Losses in strainer and bends etc.

$$= 0.25 \times h_f = 0.25 \times 3.64 = 0.91 \text{ m}$$

Total losses $= 3.64 + 0.91 = 4.55$ m

$$H = 15 + 5 + 0.29 + 4.55 = \mathbf{25.84\ m}$$

Hence, $\text{H.P.} = \dfrac{\gamma_w \cdot QH}{735\eta}$

$$= \frac{(9.81 \times 10^3) \times 4.17 \times 10^{-2} \times 25.84}{735 \times 0.65} = 22.1;\ \textbf{say 23 H.P.}$$

Hence, 23 H.P. motor is required.

16.26. Ground Water Prospecting

The term 'ground-water prospecting' means *searching for the ground water.* It not only includes to find out the places where the ground water is available, but also to find out its approximate quantity and quality as well. This job can be done *by carrying out, what are called* groundwater surveys.

These **ground water surveys** or investigations are extremely important in arid regions, where ground water is scarcely available. In such regions, if such surveys are not carried out in advance, and the excavation of wells is undertaken, then everything may come out to be futile, as no sufficient and good quality water may become available for obtaining the required water supplies.

Besides the problem of conducting such surveys for obtaining water supplies, another problem which an engineer may face is to detect whether any ground water would be encountered in underground construction operations. The engineer will also have to find out means and ways to check and control that ground water and the problems created by it.

For both these purposes, investigations would have to be conducted to detect the presence of water at the given region or at the particular site, and to fairly estimate its quality or quantity, or both.

The very first indicator of the presence of groundwater in an arid region, is the presence of *plants and vegetations*, especially the plants that habitually grow in arid regions only when they can send their roots down to the watertable. Such plants are called *phreatophytes.* The type of plant, will also to certain extent, indicate the depth of the watertable. The plants, may also to some extent indicate the quality of the ground water. An idea about the different types of plants growing in a particular region, and their peculiar indications about the presence of ground water, can be gathered from the observant local inhabitants. This type of investigation is purely preliminary, and must be followed and confirmed by *geological* and other *geophysical* surveys and field investigations.

The geological studies by which groundwater may be investigated would comprise of making a detailed *geological mapping* of the area, which may give an accurate

knowledge of the existing geological structures, including the lithological character, mode of origin, and petrological features of various underground rocks and strata.

In addition to these usual investigations, however, close attention must be paid to all evidences of ground water. Say for example, records of all the existing wells and bore holes must be examined and correlated. New test boreholes may also be excavated and examined, if needed. By such means, when records become available, it is possible to prepare *contour maps for the groundwater levels* over large areas of the country. These groundwater contour maps provide a means of fairly estimating the quantity of available ground water. This quantity of available ground water may also be estimated by some hydrological methods, which depend on studying the relationships between determing and using the *specific yield* factors for the rocks in question.

'Specific yield' as defined earlier, is the volume of water (expressed as percentage of the total volume of the aquifer) which will be drawn freely from the aquifer. In other words, it will be slightly less than the porosity, as some water is retained in the aquifer by molecular or capillary forces. For the measurement of specific yield, various methods have been suggested; some of which can be utilised in a laboratory on samples of material encountered in the area under investigation, and testing them for their permeability, porosity, etc. Other methods necessitate field observations and pumping tests in these bore holes, to finally calculate the specific yields.

Shock waves or seismic waves produced by artificial explosions have also been used these days, to detect the presence of ground water and its depth of occurrence. The principle underlying these seismic methods is that the velocity of the elastic waves (caused by artificial shocks) is different in moist deposits than in the dry formations of the same composition. These *seismic surveys* are, therefore conducted by firing a charge of explosive, near the ground surface, and recording the arrival of the resulting shock waves at a series of geophones, remote from the shock point. Since the velocity of the shock waves depends on the type of formation and the presence of water, it may be possible to detect and estimate the depth of the watertable from the differences in the indicated velocities recorded at several geophones.

The other important type of geophysical investigation which may be performed for groundwater exploration is called the *resistivity surveys.*

Resistivity surveys make use of the fact that water increases the *conductivity* of the rocks, and thereby decreasing their *resistivity*. Hence, if it can be established geologically, that the same rock formation is existing for a certain depth, say 100 m, and by electrical testing it is found that the resistivity is decreasing below say 60 m depth, then it can be easily concluded that the water is present below 60 m depth.

The *resistivity of rocks at various depths* can be calculated on the following principle : If electrodes (generally four) are inserted in the ground and connected in a circuit to a source of electrical energy, the current will flow from one electrode, pass through the ground, and finally leave through the other electrode. The depth to which this current penetrates in the ground, depends upon the distance between the two outer electrodes (it is generally of the order of $\frac{1}{4}$th the distance between electrodes). Thus, it is possible to send the current deeper into the ground by simply increasing the distance between the electrodes (Refer Fig. 16.62). Hence, it should be possible to determine the resistivity of the given formation-rocks, by measuring the passing current in the potentiometer circuit ; and at different depths, by repeating the experiments with different electrode

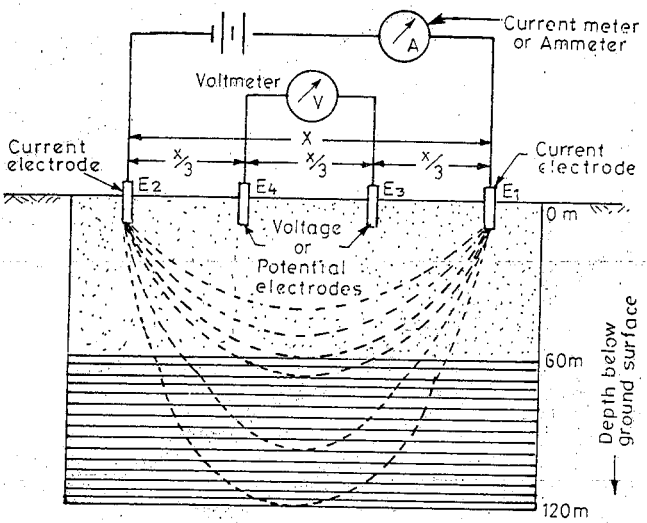


Fig. 16.62. Wenner's Resistivity method for Ground water exploration.

$$\text{Resistivity} = \frac{\text{Voltage}}{\text{Current}} \times 2\pi\left(\frac{x}{3}\right)$$

spacings. The resistivity variations with depth can then be plotted easily, and the variations studied along with geological or stratigraphical* knowledge of the existing rock formations, as pointed out above.

Say for example, in Fig. 16.63, the resistivity is decreasing below 60 m depth and up to 120 m depth; whereas let the geology of the area confirms the existence of the same type of rock (say Sandstone) up to this 120 m depth; then it easily gives the inference that water is existing between 60 m and 120 m depth.

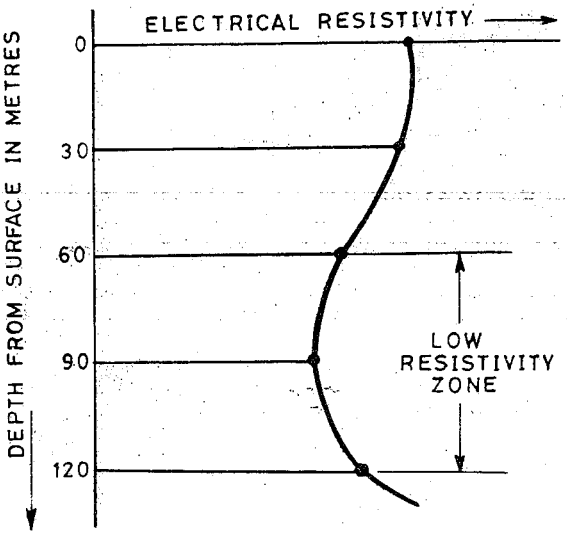


Fig. 16.63. Variation of Resistivity with depth.

Both seismic and resistivity surveys should be made and interpreted by persons who are fully trained in their work. In fact, neither of these two methods, specifically locates the groundwater, but merely indicate discontinuties which may bound an aquifer. With a few test holes as control points, however, large areas may be surveyed quite rapidly by seismic or resistivity methods. The resistivity data may also give an indication of the chemical quality of the groundwater, since dissolved salts reduce the resistivity of water.

* historical geology.

Electrical testing done in oil wells, and their recording called *electrical logs* or *resistivity logs*, are also often useful in ground water studies.

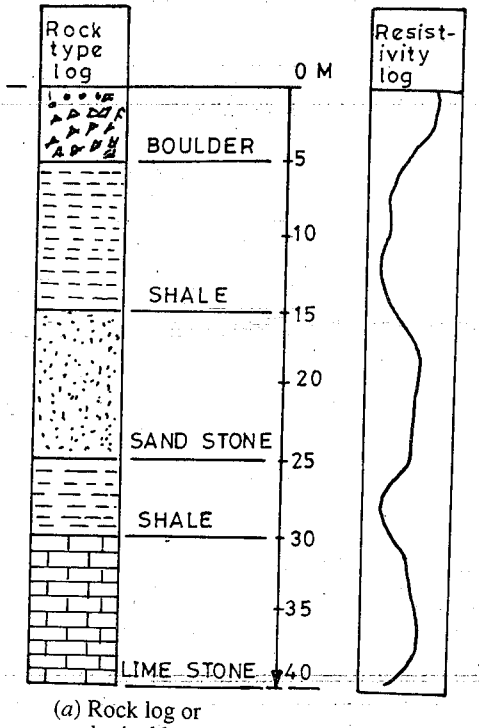
16.26.1. Logs or Recording of Bore-hole Data.

Logs may be defined as the records of the sub-surface investigations, and provide useful information regarding the nature and properties of the materials occuring at various depths below the ground surface. These records may be in the form of mere tables or graphic plots with symbolic descriptions. The data making the basis of these records may be obtained by different methods, and accordingly there are many types of bore-hole logs. A few examples of such bore hole logs are described below :

(*i*) *Geological logs* or *well logs*, as pointed out in article 16.25.6.4, are those which represent the type of the strata existing at different depths, and encountered during direct digging or boring of the wells. [Refer Fig 16.64 (*a*)]

(*ii*) *Resistivity logs* are those which indicate the values of electrical resistivity of the rocks at different depths, determined by special techniques from surface downwards in bore holes. [Refer Fig. 16.61 (*b*)].

(*iii*) *Sonic logs* are those which indicate the values of velocities of compressional waves at different depths.



(*a*) Rock log or geological log. (*b*) Resistivity log.

Fig. 16.64. Bore-hole logs.

(*iv*) *Thermal logs* are those which indicate variation in temperature with depth, as determined directly with the increasing depth.

(*v*) *Radiometric logs*, similarly, indicate the variations of radioactivity with depth.

Borehole logging plays an important role in all the major programmes of water exploration. Most of the properties, investigated for, and represented in bore hole logs, are highly sensitive to water content of the rocks, and as such, when interpreted in that light, may yield useful information.

16.27. Advantages and Disadvantages of Tube-well Irrigation Over Canal Irrigation

Advantages :

(*i*) Isolated lands which cannot be served by canals can be easily irrigated by tube-wells.

(*ii*) Cultivators can have their own private tube-wells and thus have not to depend on the government owned canal waters.

(*iii*) Canal irrigation projects require huge funds and considerable time, while wells can be constructed wherever required in a small time and with lesser funds.

(*iv*) The supply of water from a well can be started as soon as required and can be stopped at any moment, thus taking advantage of momentary rainfall.

(*v*) Tube-well channels are of shorter length and generally lined, thus resulting in lesser percolation losses.

(*vi*) Tube-well water is generally sold on a volumetric basis, which results in optimum utilisation of water at the correct time.

(*vii*) Pumping from ground water by means of tube-wells helps in lowering the ground watertable, and thus helps in reducing water logging, which on the other hand, is generally caused by canal irrigation.

(*viii*) With the help of well irrigation, more than one crop may be grown in an year.

(*ix*) The well water which is warmer in cold weather and colder in hot weather is more suitable to crops.

(*x*) The well water is more pure than the canal water, and thus the irrigation scheme may be combined with rural water supply-scheme.

(*xi*) The land acquisition is less for tube-well irrigation.

(*xii*) Unless drought continues for many years, well irrigation does not fail in droughts, while a canal supply may fail in a single drought or at the most in two or three consecutive drought years.

Disadvantages :

(*i*) If the electric supply fails (which generally does in drought years), the pumps of the tube-wells cannot be operated, and hence, the well water cannot be made available to the crops, unless diesel power is available.

(*ii*) Canal irrigation projects are generally combined with flood control and hydropower projects, thus giving added benefits.

(*iii*) The well water which is generally free from silt is not so good from manuring point of view as is the silted canal water.

(*iv*) The tube-well water proves much costlier than the canal water since the well water has to be lifted by pumps.

(*v*) The life of a tube-well is limited.

(*vi*) Frequent breakdown of power and motor parts cause large scale interruptions in the working of tube-wells. So much so that the usual running hours have been estimated to be 3,760 out of annual of 8,760.

Conclusion. Truely speaking, a combined irrigation system is the best. In other workds, we must install a large scale canal irrigation system in the country supported and supplemented by tube-wells throughout. This is what is being followed in India, especially in Punjab and U.P., where irrigation is most intensively as well as extensively practised.

PROBLEMS

1. (*a*) discuss briefly as to how the water is stored into the ground water reservoir. Briefly mention the various zones and importance of the 'zone of saturation' in this connection.

(*b*) Enumerate the different methods by which the ground water is drained and used in our country.

(*c*) What is meant by artificial recharge of ground water ? Enumerate the different methods which are used for this purpose and describe one of them briefly.

2. Define and explain any five of the following terms, as used in connection with ground water :
   (*i*) Capillary fringe
   (*ii*) Specific yield
   (*iii*) Pellecular water
   (*iv*) Field capacity
   (*v*) Permeability and transmissibility
   (*vi*) Aquifers and aquicludes
   (*vii*) Non-artesian and artesian wells
   (*viii*) Perched aquifers
   (*ix*) Storage coefficient
   (*x*) Specific capacity of wells
   (*xi*) Well loss.

3. (*a*) Distinguish between non-equilibrium and equilibrium conditions in an aquifer from which water is withdrawn through a well. Explain when the above conditions can be expected in an aquifer. (*Engg. Services, 1970*)

(*b*) Derive a formula for discharge of well in a homogeneous artesian aquifer assuming equilibrium flow conditions. (*Engg. Services, 1970*)

(*c*) In an artesian aquifer, the drawdown is 1.5 m at a radial distance of 8 metres from a well after two hours of pumping. On the basis of Theis' non-equilibrium equation, determine the pumping time for the same drawdown (1.5 metre) at radial distance of 20 metres from the well.

[**Hint.** Follow example 16.7] [**Ans.** 12.5 hours]

4. Distinguish with sketches if necessary, the difference between an unconfined and a confined aquifer. Derive a formula for discharge of a well in a homogeneous unconfined aquifer assuming equilibrium flow condition; state the assumptions on which the formula is based. (*Engg. Services, 1969*)

5. (*a*) What is Dupuit's equation ? State the assumptions that enter in its development. Explain the Theis' formula. What is well function? (*Engg. Services, 1967*)

(*b*) Explain the following terms :
   (*i*) Aquifer,
   (*ii*) Aquiclude,
   (*iii*) Capillary fringe,
   (*iv*) Equilibrium drawdown,
   (*v*) Specific yield, and
   (*vi*) Perched watertable. (*Engg. Services, 1967*)

6. (*a*) Explain the following :
   (*a*) Vadose water,
   (*b*) Pellicular water;
   (*c*) Artificial recharge;
   (*d*) Non-equilibrium equation; and
   (*e*) Coefficient of storage. (*Engg. Services, 1968*)

(*b*) A 30 cm dia well penetrates 20 m below the static watertable. After 24 hours of pumping at 5000 litres per minute, the water level in a test well at 100 m away is lowered by 0.5m, and in a well at 30 m away, the drawdown is 1 m. What is the transmissibility of the aquifer?

[**Hint.** Follow example 16.5]

7. (*a*) Explain :
   (*i*) Specific retention of a soil
   (*ii*) Specific yield of an aquifer
   (*iii*) Storage coefficient of an aquifer
   (*iv*) Specific capacity of a well. (*Engg. Services, 1972*)

(*b*) Step drawdown test was carried out in a well constructed to give a yield of 2300 litres per minute. The data obtained is given below. Determine the well loss and efficiency of the well.

Yield in l.p.m.	*Drawdown in metres*
1230	3.72
1840	5.65
2460	7.62
3070	9.62
4080	13.07
6130	20.21

(*Engg. Services, 1972*)

8. (*a*) State Dupuit's assumptions for obtaining general equations governing ground water flow. Derive an expression for the confined aquifer. How can the expression be used to evaluate the aquifer permeability?

(*b*) A well penetrating an aquifer which is underlain and overlain by inpermeable layers was tested with a uniform discharge of 1000 litres/min. The steady state drawdowns measured in two observation wells which were at 1 m and 10 m radial distances from the centre of the pumped well were 13.40 m and 4.2m, respectively. Determine the hydraulic properties of the aquifer, if its saturated thickness is 10 m.

[**Ans.** Transmissibility = 0.0398 m^2/min.
Hydraulic conductivity = 0.00398 m/min.]

9. A 10 cm diameter well was pumped at a uniform rate of 500 litres/min., while observations of drawdown were made in an observation well located at a distance of 50 m from the well. The original head of water, measured from the top of the impervious layer was 25 m. The hydraulic conductivity of the aquifer was 1.83×10^{-3} m/min. Determine the drawdown at the face of the well, using Dupuit-Thiem equation, and assuming that the flow to the unconfined aquifer is under steady state. [**Ans.** 20 m]

10. Write short notes on any four of the following:

(*i*) Infiltration wells and infiltration galleries.

(*iii*) Darcy's law for measuring velocity of ground water flow.

(*iv*) Permeability and transmissibility and their relationship.

(*v*) Measuring the yield of underground water sources.

(*vi*) Surface of seepage and free surface curve.

(*viii*) Cavity formation in dug wells.

(*ix*) Wells and Tubewells

11. (*a*) Differentiate between shallow dug wells and deep dug wells. How are dug well constructed?

(*b*) Enumerate the methods which are used for determining the yield of dug wells. Discuss briefly any one of these methods.

12. What is meant by tubewells ? What are their types ? Describe the widely used type of tubewell with a neat sketch. What are the approximate values of the average yield and depth of such a tubewell ?

13. (*a*) Enumerate the different methods which are used for drilling tubewells. Discuss any one of these methods in details.

(*b*) Discuss briefly the design principles involved in the design of a strainer type of a tubewell.

(*c*) What is the average life of tubewells and what are the reasons for their failure ? What remedies will you suggest for increasing their life ?

14. Write short notes on any four of the following :

(*i*) Shallow and Deep tubewells;

(*ii*) Artificial recharge of groundwater;

(*iii*) Groundwater prospecting;

(*iv*) Development of tubewells;

(*v*) Well screens;

(*vi*) Gravel pack slotted pipe tubewells;

(*vii*) Design of gravel pack;

(*viii*) Surface and subsurface sources of water.

17

Dams in General and a few Dams in Particular

17.1. . General

17.1.1. Definition. A dam may be defined as an obstruction or a barrier built across a stream or a river. At the back of this barrier, water gets collected, forming a pool of water. The side on which water gets collected is called the upstream side, and the other side of the barrier is called the downstream side. The lake of water which is formed upstream is often called a *reservoir.*

17.1.2. Uses. The water collected in this lake can be used for recreation, or as a reservoir of drinking water, or it can be tapped off into surrounding farm land for irrigation. The energy of this collected water can be used to turn a mill to grind wheat or to turn the blades of a tubrine to generate electrical power. And in times of floods, the dams can serve as protections for the towns and cities farther down the river.

Apart from these numerous advantages and uses (such as navigation, irrigation, electricity, flood control, etc.) of a dam, it sometimes helps us in planning war strategy and helps us in controlling the advancement of enemies and their forces. Dams have been frequently opened in times of war. The Dutch breached their dikes during Second World War to bedevil the invading Germans. Chinese used to destroy their dikes to flood out the enemy. Russian army retreating from the Nazi marauders, partly destroyed the famous Dneprostroi Dam in the Ukraine to keep its power plant from falling into the hands of Hitler's men.

17.1.3. History of Dam Construction. It is very difficult to say as where and when the first man-made dam was built. Archaeological evidences help in estimating that the very first man made dam is at least 3000—5000 years old. Whenever it was built, that first dam was almost certainly an irrigation dam. Its designer might have observed beavers at work or he might have thought of it the some other way. Beavers are mammel creatures belonging to the family of rats, mice, squirrels. They live under water and are generally 1.2 m long including more than 0.3 m of tail, and weigh upto 25 to 30 kg. These rodents produce fur. These creatures create dam type barriers to create a place where their family can live. They provide themselves with comfortable ponds to live in, by building dam type barriers across the stream with the help of trees which they themselves cut.

These elegant structures, built out of logs, buttressed with twigs and branches and sealed with mud and stone help us many a times in controlling silt entry into the stream. A beaver dam accumulates silt brought down by its stream. When a beaver dam is breached, the silted water pours through, and the fertile silt is deposited over the wide area. This creates what farmers call a *beaver's meadow,* where crops grow particularly well.

Beavers are sometimes encouraged to build dams in areas where man can not reach easily to construct man-made dams.

17.1.4. Modern Dams. The first modern dam of the world was perhaps constructed on the Nile river, in Egypt at *Aswan.* It was completed in 1902 and was a major engineering project.

This famous **Aswan Dam** was designed primarily to control the flooding of the Nile river, to promote irrigation in the Nile Valley, and to further navigation along the river.

Aswan is 1200 km from the mouth of the Nile. The site was chosen because the river at Aswan is shallow and has a granite bed, on which a firm foundation could be erected.

The first step in damming any major river is to divert the flow of water so as to permit dam construction. This was accomplished at Aswan, by constructing a circular earthen barrier around the area chosen from the dam foundations. At low river, this enclosure was pumped out. All the work had to be completed before the river flooded again. The foundation was laid round the clock with a huge labour force. So much so that 3,600 tons of masonry were put in places in a single day.

This 120′ high dam, running for about $1\frac{1}{2}$ miles from shore to shore, 100 feet thick at its base and 24′ wide at top, costed about 1.5 crore dollars, when completed in 1902. A roadway ran along the upper rim. About 1 billion tons of water was its storage capacity, and 180 sluices were constructed into this barrier. Water could be collected during the rainy season and released through the sluice gates during the summer and drought.

Roosevelt Dam. The next famous dam of the world was completed in 1911 on the Salt River of Arizona (U.S.A.) and was called Roosevelt dam.

The Aswan dam was constructed with stone, while the Roosevelt dam was constructed with solid blocks of concrete : which was of the type known as 'Solid Masonry Gravity Dam' : which simply means that it was built with solid blocks of concrete, which hold back the flow of water by sheer weight. This type of dam is one of the most ancient.

The construction was started in 1905. Despite the numerous troubles in its construction, the dam was completed in 1911 with a reservoir capacity of $\frac{1}{2}$ billion gallons of water. The completed dam, 280 feet high, was *158 feet thick* and spanning for a length of 1,125 feet. Had the dam been built without the benefit of 19th century engineering advances in design and stress control, it would have had to be 700 to *1000 feet thick* at base ; a far more expensive and cumbersome thing to build.

This dam was a fascinating advancement, but was soon overshadowed by other vastly greater dams, such as Hoover Dam, (726′ high), Bhakra Dam (740′ high), etc. All of them are 'solid concrete gravity type' dams.

17.2. Various Kinds of Dams

Before we describe some of the famous dams of the world, it is worth while to classify the various types of dams.

Most engineers recognise seven general types of dams. Three of them are ancient in origin, and four have come into general use only in the last about 100 years or so.

The three older types of dams are :

(1) Earth Dams

(2) Rook-fill dams

(3) Solid masonry gravity dams.

These types are discussed below :

(1) **Earth Dams.** Earth dams are made of soil that is pounded down solidly. They are built in areas where the foundation is not strong enough to bear the weight of a concrete dam, and where earth is more easily available as a building material compared to concrete or stone or rock.

Some important earth dams of the world are :

(*i*) Green mountain dam on Colorado river in U.S.A.

(*ii*) Swift dam in Washington in U.S.A.

(*iii*) Side flanks of Nagarjun Sagar dam in India.

(*iv*) Trinity Dam in California in U.S.A.

(*v*) Maithan Dam in India (which is partly Earthen and partly Rockfill).

(2) **Rockfill Dams.** Rockfill are formed of loose rocks and boulders piled in the river bed. A slab of reinforced concrete is often laid across the upstream face of a rockfill dam to make it water-tight.

Some important rock-fill dams of the world are ;

(*i*) The Salt Springs Dam in California (345′ high) in U.S.A.

(*ii*) The San Gabriel No. 1 Dam (321′ high) in U.S.A.

(*iii*) Cougar Dam on Mc-Knezie River in Oregon (445′ high) in U.S.A.

(3) **Solid-masonry Gravity dams.** These are familiar to us by now, after we have talked about Aswan, Roosevelt, Hoover, and above all Bhakra dam.

These big dams are expensive to be built but are more durable and solid than earth and rock dams. They can be constructed on any dam site, where there is a natural foundation strong enough to bear the great weight of the dam.

These three types of dams were all found in ancient days. In recent times, *four other types of dams have come into practice.* They are :

(4) Hollow masonry gravity dams ;

(5) Timber dams ;

(6) Steel dams ; and

(7) Arch dams.

(4) **The hollow masonry gravity dams.** These are essentially designed on the same lines on which the solid masonry gravity dams are designed. But they contain less concrete or masonry ; about 35 to 40% or so. Generally, the weight of water is carried by a deck of R.C.C. or by arches that share the weight burden. They are difficult to build and are adopted only if very skilled labour is easily available, otherwise the labour cost is too high to build its complex structure.

(5) **Steel dams.** These are not used for major works. Today, steel dams are used as temporary coffer dams needed for the construction of permanent dams. Steel coffer dams are usually reinforced with timber or earthfill.

(6) **Timber dams.** These are short lived, since in a few years time, rotting sets in. Their life is not more than 30 to 40 years and must have regular maintenance during that time. However they are valuable in agricultural areas, where a cattle raiser may need a pool for his live stock to drink from, and for meeting other such low-level needs.

(7) **Arch dams.** Arch dams are very complex and complicated. They make use of the horizontal arch action in place of weight to hold back the water. They are best suited at sites where the *dam must be extremely high and narrow.* Some examples are :

(*i*) *Sautet dam* on the Drac River in France, 414′ high, but only 230′ long at top and 85′ long at bottom of the gorge, 56′ thick at bottom and 8′ thick at top.

(*ii*) *The Tignes dam* in France (592′ high).

(*iii*) *Mauvoisin dam* on the Drause River in Switzerland, (780′ high).

(*iv*) *Idduki* dam in Kerala State, across the Periyar river, which is the only arch dam in India. It is 366 m (1200′) long double curvature arch dam, made in concrete, and has a height of about 170 m (560′).

17.3. Problems in Dam Construction

Dams are extremely useful things. Anyone who lives in Punjab or at Asansol in West Bengal, knows how valuable dams are. The farmers of Punjab and people getting electricity from the Bhakra sing praises for it. The people of areas benefitted by various dams and other ancillary works on Damodar river are really thankful to those human beings who have miraculousy harnessed the Damodar river for them. The prosperity and welfare of millions and billions of people depend directly on these towering handsome dams with which the nation's rivers have been harnessed.

But dams can cause problems too. Dams have drawbacks and disadvantages also. Let us here discuss some of the negative features of dams and let us see what can be done to overcome them. There are four major problems, in general, which are posed by such huge constructions. They are:

(1) Fish Problem ;

(2) Submergence Problem ;

(3) Failure Problem; and

(4) Bomb Problem

They are described below:

(1) **Fish Problem.** On large rivers, in late summer season, fish move from downstream to upstream to lay their eggs. These eggs are fertilised by male fish. The old fish may get exhausted and the new born fish again move downstream. They, after two to three years, return to their ancestral spawning place and may die after getting exhausted, while the newborns move downstream. The cycle goes on for years.

The fish which move to their ancestral spawning place (upstream) are called *anadromous fish. Salmon and Hilsa* are typical examples of such a fish. These are commercially valuable fish, and important industries are dependent on them.

When a dam barrier is constructed on a river, these fish can not move upstream to lay their eggs; because it is impossible for these fish to overtop such a barrier. But surprisingly, even when they find a barrier in their path of advancement towards their ancestral spawning ground, these fish do not return to their downstream dwelling place (*i.e.* sea). However, they go on fighting against the barrier, trying furiously to overtop it, till they get exhausted and die down. This results in a serious large scale killing of fish, causing great damage to fish industry and economy of the nations.

In the beginning, much attention was not paid to this problem; but a little later, it was realised, and serious attempts were made to find out solution to the problem.

Sometimes, fish were trapped on one side of the dam and passed on to the other side by giant steel and plastic nets. An external arrangement called *Fish Ladder* was also devised.

Fish Ladder. Just as river-going vessels can bypass a dam by using a navigation lock, so a series of 'locks' enable the fish to get over the dam. A separate channel is created, consisting of a series of little dams that form a row of pools, rising up over the big dam to reservoir level. The salmon, entering the lowest rung of the ladder at the base of the dam, could leap from pool to pool until they had crested the dam. Then, they could continue on through the reservoir to the spawning grounds. The new born fish called *finger lings* could later return to the sea (downstream) in the same fashion via the ladder. A section, plan and photographic view of a fish ladder has already been shown in the chapter on Weirs.

In the beginning, the fish ladders worked better in theory than in practice. The fish seemed to prefer to mill ground in splashing water under spillway, instead of entering the ladder. This difficulty was overcome by careful design that put the fish ladder in the place where it was most likely to attract the fish. Another problem was that the slow moving water was stranger to fish and they tended to collect in the lower pools without going onward.

Millions and billions were spent into fish-ladder research. Improvements in design made the fish ladder more attractive to fish, more like the rapids they were accustomed to.

Fish ladders are not always practicable from engineering stand point. In such cases, other steps have to be taken to protect the fish.

Meanwhile, other experiments are going forward to see if fish can be successfully induced to spawn in waters other than their own ancestral spawning grounds. In the long run, it may save millions of currency to construct *fish hatcheries* instead of *fish-ladders.* There are many possible solutions to the problem of anadromous fish, and research is being undertaken in different regions of the world to find out a better solution to the problem.

(2) **Submergence Problem.** Whenever a dam is constructed across a river to store water on the upstream side, a large area gets submerged due to the rise in the water levels. The entire area which gets submerged, forming a reservoir, has to be calculated and acquired before a dam can be constructed. The owners of the land have to be persuaded, adequately compensated, and well settled somewhere else, before, the work can be taken up in hand. Hence it is necessary to investigate the probable damage caused by this submergence.

(3) **Failure Problem.** We try our best to build dams to last as long as possible. Every person whosoever has worked on a dam hopes that the dam will live as long as the pyramids of Egypt. But many a times, the dam give way under the continued insistent pressure of the water penned up behind them. This failure of the dam may be caused either due to bad workmanship or due to faulty design or due to the occurrence of unanticipated floods.

Luckily, these disasters have been comparatively rare in this century. Dams used to give way easily in olden times, but due to engineering advancement in modern times, their failure has been considerably reduced.

These huge structures are now properly designed, keeping in view the various forces which they are going to face. Proper and rational design, good supervision and constant vigil and watch during maintenance period ensures their safety and makes us fairly confident of it. Bhakra Dam on Satluj River in India and Boulder Dam on Colorado River in U.S.A. *cannot fail* in one attempt, how furiously these rivers may try to move their foundations. We are fairly confident of this, but sometimes the confidence is *rudely and cruelly repaid with tragedies.*

Dams may sometimes fail due to excessive and unanticipated earthquakes. The *Koyna Dam* in India was at the verge of failure in 1968 earthquake. Thanks to the efforts of the Indian engineers who saved that dam by toiling hard day and night. A very confident dam called *Vega de Tera Dam* is Spain failed in January 1959. The people were tucked in the town of Rivaldelago. The disaster caused was tremendous. Rivaldelago was flattened. Telephone poles were snapped like matchsticks. Within moments, 123 villages were drowned. Several hundred luckier ones were saved, but were rendered homeless. This was a case where a dam had simply not been built strong enough to bear the full weight of its intended reservoir. Heavy rains wrecked it. *Faulty design and bad engineering must be blamed.*

Another important dam called *The Malpasset Dam,* a 200 feet high arch dam on the Reyran River, was completed in 1954. This dam gave way in December 1956, causing 421 persons to die in floods. Investigations revealed that the dam had failed because the foundation rock has shifted along a thin clay seam in the left abutment, making the dam unstable and vulnerable to any serious stress.

We learnt from our mistakes;and several other dams of the same type, then under construction in Europe, were quickly resurveyed to find the possibility of such a geological formation. This was very very small comfort to the relatives of those who died when Malpasset failed; but at least, we should learn from our mistakes and there should be no such repetitions.

(4) **The Bomb Problem.** The dams create dangers in wars, especially in modern atomic age. One single atom bomb may cause the failure of Hoover Dam (Boulder Dam) or Bhakra Dam. The resultant failure of such a dam will create catastrophes, but also, it will get contaminated by radioactivity from which there could be no escape.

This is an important point which is generally stressed by opponents of big dams. But the only answer to this argument is that it would not be advisable to deprive ourselves of the benefits of big dams simply because they are hazards in war time. After all, an atom bomb dropped in Calcutta, Delhi, or New York would also cause tremendous damage and catastrophe, but this *does not mean* that we should not *develop big cities.*

Atomic war is dangerous to every aspect of living and not only to the construction of dams. We don't refuse riding in automobiles or aeroplanes because of the fear of accidents. *Certain risk has to be accepted if there is to be progress.*

So, without denying the very great damage that could be caused by atomic explosions at our dams, we must go on building dams. We need them and we must devote our energies to the cause of continued peace, so that bombs will never be able to fall. We may also take more precautions, and anticraft guns and radars can be established at and in the vicinity of such important works. The use of atomic energy for peaceful purposes and a general feeling of brotherhood is the only possible way to reduce such threats.

17.4. Selection of the Type of Dam and Their Classifications

(1) *Classification.* Dams can be classified in various ways depending upon the purpose of the classification.

(1) *Classification According to the Material used for Dam Construction :*

The dams classified according *to the material used* for construction are: Solid masonry gravity dams, Earthen dams, Rockfill dams, Hollow masonry gravity dams, Timber dams, Steel dams, and R.C.C. Arch dams. They have already been explained in a previous article.

(2) *Classification According to Use*

(*i*) **Storage Dams.** They are constructed in order to store water during the periods of surplus water supply, to be used later during the periods of deficient supply. The stored water may be used in different seasons and for different uses. They may be further classified depending upon the specific use of this water, such as navigation, recreation, water supply, fish, electricity, etc.

(*ii*) **Diversion Dams.** These small dams are used to raise the river water level, in order to feed an off-taking canal and or some other conveyance systems. They are very useful as irrigation development works. A diversion dam is generally called a **weir** or a **barrage.**

(*iii*) **The Detention Dams.** They detain food-waters temporarily so as to retard flood runoff and thus minimise the bad effects of sudden flood.

Detention dams are sometimes constructed to trap sediment. They are often called debris dams.

(3) *Classification According to Hydraulic Designs*

(*i*) **Overflow Dams.** They are designed to pass the surplus water over their crest. They are often called Spillways. They should be made of materials which will not be eroded by such discharges.

(*ii*) **Non-overflow Dams.** They are those which are not designed to be overtopped. This type of design gives us wider choice of materials including earthfill and rockfill dams.

Many a times, the overflow dam and the non-overflow dam are combined together to form a composite single structure.

(*iii*) **Rigid Dams and Non-rigid Dams.** Rigid dams are those which are constructed of rigid materials like masonry, concrete, steel, timber, etc.; while non-rigid dams are constructed of earth and rock-fill. They have already been explained.

17.5. Factors Governing the Selection of a Particular Type of Dam

Whenever we decide to construct a dam at a particular place, the first baffling problem which faces us, is to choose the kind of the dam. *Which type will be the most suitable and most economical?* Two, three kinds of dams may be technically feasible, but only one of them will be the most economical. Various designs and their estimates have to be prepared before signalling one particular type. The various factors which must be thoroughly considered before selecting one particular type are described below:

(1) **Topography.** Topography dictates the first choice of the type of dam. For example:

(*i*) A narrow U-shaped valley, *i.e.* a narrow stream flowing between high rocky walls, would suggest a concrete overflow dam.

(*ii*) A low, rolling plain country, would naturally suggest an earth fill dam with a separate spillway.

(*iii*) The availability of a 'Spillway Site' is very important while selecting a particular kind of a dam.

(*iv*) A narrow V-shaped valley indicates the choice of an arch dam. It is preferable to have the top width of the valley less than one-fourth of its height. But a separate site for the spillway must also be available.

(2) **Geology and Foundation Conditions.** The foundations have to carry weight of the dam. The dam site must be thoroughly surveyed by geologists, so as to detect the thickness of the foundation strata, presence of faults, fissured materials, and their permeability, slope, and slip, etc.

The various kinds of foundations generally encountered are discussed below:

(*i*) *Solid Rock Foundations.* Solid rock foundations such as granite, gneiss, etc. have a strong bearing power. They offer high resistance to erosion and percolation. Almost every kind of dam can be built on such foundations. Sometimes, seams and fractures are present in these rocks. They must be grouted and sealed properly.

(*ii*) *Gravel Foundations.* Coarse sands and gravels are unable to bear the weight of high concrete gravity dams and are suitable for earthen and rock-fill dams. Low concrete gravity dams up to a height of 15 m may also be suggested on such foundations.

These foundations have high permeability and, therefore, subjected to water percolation at high rates. Suitable cut-offs must be provided to avoid danger of undermining.

(*iii*) *Silt and Fine Sand Foundations.* They suggest the adoption of earth dams or very low gravity dams (upto height of 8 m). A rockfill dam on such a foundation is not suitable. Seepage through such a foundation may be excessive. Settlement may also be a problem. They must be properly designed to avoid such dangers. The protection of foundations at the downstream toe from erosion must also be ensured.

(*iv*) *Clay Foundations.* Unconsolidated and high moisture clays are likely to cause enormous settlement of the dam. They are not fit for concrete gravity dams or for rock-fill dams. They may be accepted for earthen dams, but that too, after special treatment. (*v*) *Non-uniform Foundations.* At certain places, a uniform foundation of the types described above may not be available. In such a case, a non-uniform foundation of rock and soft material may have to be used if the dam is to be built. Such unsatisfactory conditions have to be dealt with by special designs. However, every problem is an individual problem and a solution has to be found by experienced engineers. For example—

The Jawahar Sagar Dam in Rajasthan offered such a problem. A bed of clay was encountered, between the base of the dam and solid rock foundation. It was not economically feasible to remove this clay bed. The solution adopted was to anchor the base of the dam to the foundations below, by means of prestressed cables.

(3) **Availability of Materials.** In order to achieve economy in the dam, the materials required for its construction must be available locally or at short distances from the construction site.

Sometimes, good soil is easily available, which naturally calls for an earthen dam. If sand, cement and stone, etc., are easily available, one should naturally think of a concrete gravity dam. If the material has to be transported from far off distances, then a hollow concrete dam (Buttress) is a better choice.

(4) **Spillway Size and Location.** Spillway, as defined earlier, diposes of the surplus river discharge. The capacity of the spillway will depend on the magnitudes of the floods to be by- passed. The spillway will, therefore, become much more important on streams with large flood potential. On such rivers, the spillway may become dominant structure, and the type of dam may become the secondary consideration.

The cost of constructing a separate spillway may be enormous or sometimes a suitable separate site for a spillway may not be available. In such cases, combining the spillway and the dam into one structure may be desirable, indicating the adoption of a concrete overflow dam.

At certain places, where excavated material from a separate spillway channel may be utilised in dam embankment, an earthfill dam may prove to be advantageous. Small spillway requirement often favours the selections of earth fill or rockfill dams even in narrow dam sites.

The practice of building a concrete spillway on earth and rock embankments is being discouraged these days, because of their conservative design assumptions and the vigil and watch that has to be kept during their operations.

(5) **Earthquake Zone.** If the dam is to be situated in an earthquake zone, its design must include the earthquake forces. Its safety should be ensured against the increased stress induced by an earthquake of worst intensity. The type of structures best suited to resist earthquake shocks without danger are earthen dams and concrete gravity dams.

(6) **Height of the Dam.** Earthen dams are usually not provided for heights more than 30 m or so. Hence, for greater heights, gravity dams are generally preferred.

(7) **Other Considerations.** Various other factors such as, the life of the dam, the width of the roadway to be provided over the dam, problem of skilled labour, legal and aesthetic point must also be considered before a final decision is taken. Overall cost of construction and maintenance and the funds available will finally decide the choice of a particular kind of a *dam at a particular place.*

17.6. Selection of Dam Site

The selection of a site for constructing a dam should be governed by the following factors:

(1) Suitable foundations (as determined in the previous article) must be available.

(2) For economy, the length of the dam should be as small as possible, and for a given height, it should store the maximum volume of water. It, therefore, follows, that the river valley at the dam site should be narrow but should open out upstream to provide a large basin for a reservoir. A general configuration of contours for a suitable site is shown in Fig. 17.1.

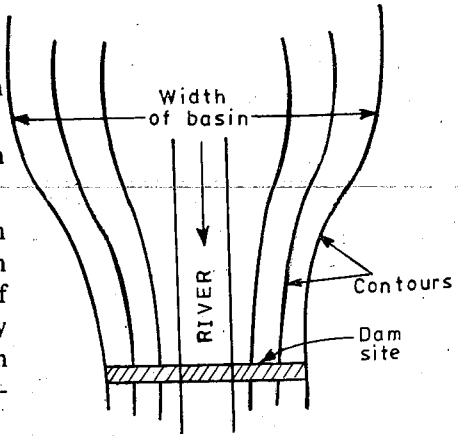


Fig. 17.1

(3) The general bed level at dam site should preferably be higher than that of the river basin. This will reduce the height of the dam and will facilitate the drainage problem.

(4) A suitable site for the spillway should be available in the near vicinity. If the spillway is to be combined with the dam, the width of the gorge should be such as to accommodate both.

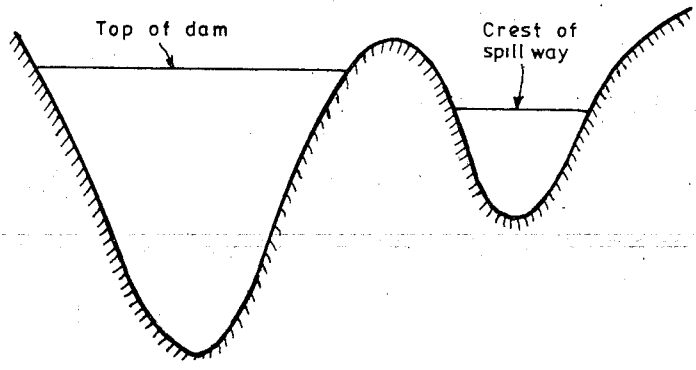


Fig. 17.2

The best dam site is one, in which a narrow deep gorge is separated from the flank by a hillock with its surface above the dam, as shown in Fig. 17.2.

If such a site is available, the spillway can be located separately in the flank, and the main valley spanned by an earthen or similar dam. Sometimes, the spillway and concrete masonry dam may be compositely spanned in the main gorge, while the flanks are in earth at low cost.

(5) Materials required for the construction should be easily available, either locally or in the near vicinity, so that the cost of transporting them is as low as possible.

(6) The reservoir basin should be reasonably water-tight. The stored water should not escape out through its side walls and bed.

(7) the value of land and property submerged by the proposed dam should be as low as possible.

(8) The dam site should be easily accessible, so that it can be economically connected to important towns and cities by rails, roads, etc.

(9) Site for establishing labour colonies and a healthy environment should be available in the near vicinity.

STORIES OF A FEW IMPORTANT DAMS

Before we take up the actual planning and design of concrete gravity dams and earthen dams in subsequent chapters, let us narrate the stories of certain such important dams. This will give us an idea as to what actually happens in the field and to what kind of difficulties are encountered and how they are overcome.

17.7. Hoover Dam

Hoover dam (Fig. 17.3) is a concrete gravity dam, constructed on the Colorado river in California (U.S.A.). The construction of this dam was taken off the drawing boards on January 26, 1892 when the Colorado River Commission presided by Mr. Harbert Hoover discussed its construction in their first meeting. But the actual construction of this dam could start only in late 1930. The construction took about 2 years of non-stop work, every minute of the hour, 24 hours a day, and 365 days a year. On the day of Christmas, the pouring of concrete was tremendous even under searing desert sun by day and under floodlights at night.

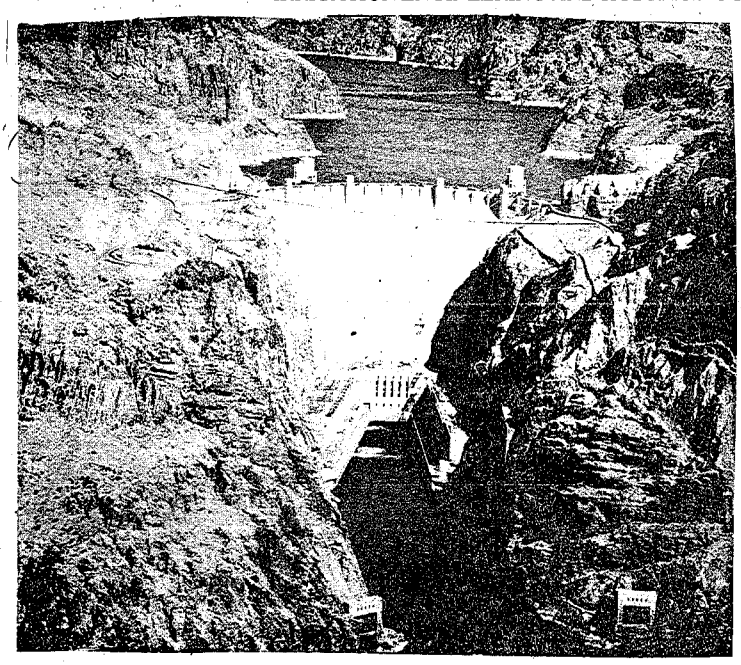


Fig. 17.3. Photo view of Hoover Dam.

Name Problem. The name of the Hoover Dam became a political issue. It was first of all named as *Boulder Dam.* Later when Herbert Hoover became the President of America, its name was changed to *Hoover Dam.* In 1933, when the Democrats replaced the Republicans, President Roosevelt changed this name again to *Boulder Dam.* It was known as *Boulder Dam* for 14 years. When again in 1946, Republicans came to power, the name was once again changed to *Hoover Dam,* which still exists today.

Dimensions of the Dam. The original dimensions of the dam are given below in F.P.S. units:

Height	= 726 ft
Span	= 1300 ft from rim to rim of the Canyon
Thickness at the base	= 660 ft
Thickness at the top	= 45 ft
Name of the reservoir formed at the back of the dam	= Lake Mead
Length of the reservoir	= 115 miles.

Something about the river. This big dam straddles the Colorado river. The Colorado river rises in the State of Colorado, runs down through Utah and into Arizona, then California, emptying finally into the Gulf of California in Mexico.

The Colorado is a river with muscles. It is a very strong river - containing huge amounts of silt and mud in it. It cuts numerous deep narrow canyons while it flows. The mile deep walls of the Grand canyon proves the power of this river.

It is a young river and all such rivers have strong powers for forming deep narrow gorges (*i.e.* canyons). They are the most turbulent, and the deepest digging. They are the best sources of hydroelectric power. Irrigation or navigation is not possible on this kind of rivers, unless, they are tamed.

Attempts were made to tame this river by constructing dikes or levees, but failed.

Spring floods of 1904 brought havoc. Another catastrophe occurred in 1916. Gila River, a tributary of Colorado, flooded at a rate of about 5, 560 cubic metres/sec. The town of Yuma in Arizona, where the Gila empties into the Colorado, was submerged to a depth of 1.2 metres.

It was extremely desirable to tame this river. But how? A solution was dreamt of. The idea of constructing a big dam was visualised by Arthur Davis. It was an ambitious plan.

Planning for the Dam. A scheme was planned. A concrete gravity dam, of about 700 to750 feet height was thought. A reservoir at the back of the dam could hold every drop of water, the Colorado could send in any 2 years of steady flow. It was to be the biggest man-made lake in the world. It was going to be a multi- purpose dam, generating about 6 billion Kilo watt-hours of electrical energy each year for the growing cities of southern California. The reservoir would hold the flood waters, and the spillways would release the required amounts of water to the downstream farmers.

Flood control, irrigation and electricity were the three main purposes of this project. Supply of drinking water to 13 cities of California and creation of a recreational and navigable lake reservoir were the additional advantages.

Selection of a suitable site. A thorough search was made for the spectacular gorges of the Colorado, seeking the best possible site. They studied 70 such sites before choosing 'Black canyon' on the border between Arizona and Nevada, 48 km from the city of Las Vegas (Nevada).

The preliminary survey of *Black canyon* took about 3 years. Here, the river flows through cliffs 1000-2000 feet (300 m to 600 m) high. At the water line, the rock walls were 350 feet (105 m) apart.

Engineers roamed in heat and sun, testing the rocks, drilling into it, to make sure that it could stand the burden of enormous weight of the concrete that would be laid upon it. *Their conclusion was that it could. They recommended that the giant dam be built.*

After the technical green-signal was obtained, some political issues such as to who will be benefitted and up to what extent, were settled with a great difficulty. The rift between the different States always persists in such huge projects.

Construction of the Dam. The real work began in late 1930. Herbert Hoover was the President of America at that time. He himself was an engineer. The work started under his vigilance, Arthur Powell Davis, 70 years old and about to retire, saw his life long dream fulfilled as he stood high above the Colorado and watched thousands of workers working hard with picks and dynamite far below.

The dam site was very hot. It was not a congenial surroundings to work. But a *dam had to be built.*

First of all, a town was built to house the workers. A permanent city, now called Boulder-city, was settled at an expenditure of about $70 million. This was spread in an area of about 300 acres and could accommodate 5, 000 workers.

Whenever a dam is constructed, the water of the river is first of all to be diverted so that the construction could start. This is called by engineers as 'to turn the river off.' 4 tunnels, each 56 feet wide and 4000 feet long were dug into the solid rock of the canyon walls. These *bypass tunnels* received the flow of the river and carried it down

to the downstream, to a point beyond the construction site. About 1 $\frac{1}{2}$ million cubic feet of rock had to be removed for building these four tunnels.

Then, the Coffer dam was built. A coffer dam is a temporary retaining embankment upstream of the site. Huge amounts of rock and earth was heaped up, forcing the river into four bypass tunnels. The bed of the river was thus laid bare.

Workers then descended into this river bed to lay the foundations for the dam. Seven million tons of concrete had to be laid. It was 660 feet thick at the base.

The dam curves upstream, so that the water load is held back in part by the walls of the canyon. The completion of the dam took about 2 years of non-stop work.

After completion, the bypass tunnels were blocked up and the water started coming and collecting against the dam. The lake formed on the upstream side was called *Lake mead.* Electrical power houses, which are of the size of 20-storey sky-scrapers were constructed. The lake, the power houses, the dam galleries etc. are open for visitors and for inspection. It was a great achievement indeed.

17.8. Bhakra Dam

Bhakra Dam is a concrete gravity dam. It is 740 feet **(226 m)** high, spanning the V-shaped gorge in the lower Shivalik hills. The dam is 1700 ft long at the top and only 325 ft at the bottom. The thickness of the dam at foundations is 1320 ft and it tapers to 30 ft at the top where a road runs. *Bhakra dam was the highest concrete gravity dam of the world when built, thus surpassing the existing* 726 ft (221 m) high Hoover dam. But the highest concrete gravity dam of the world, at present, is **Grand Dixence dam** in Switzerland **(284 m high).** Bhakra dam is situated in Himachal Pradesh State of India near a village used to be called Bhakra. It has been constructed on Satluj river. Satluj is a river coming from Himalayas. It is a perennial river but carries enormous water during floods and rains.

Downstream and upstream views of Bhakra dam are shown in Photo Fig. 17.4.

This dam has given tremendous prosperity to India and has given her a high name in the world. The various functions served by this dam are:

(*i*) Flood control (*ii*) Irrigation

(*iii*) Electricity (*iv*) Fish development.

Bhakra Project is not a single Bhakra Dam but consists of the following:

(*i*) 740 ft high Bhakra Dam (*ii*) 95 ft high Nangal Dam

(*iii*) Nangal Hydel Channel (*iv*) Ganguwal and Kotta Power houses

(*v*) Bhakra Canal System.

Planning and Construction. The survey works for the construction of this multi-purpose project started in 1919. From 1919 to 1930, the survey continued and various sites were considered for various purposes. From 1932 to 1946, the work was inter-rupted, and finally in 1946, a railway line was spread in this area, and then the actual construction started.

After selecting a suitable narrow canyon for the construction of Bhakra Dam, the dam site was dewatered after the river was 'turned off'. Two diversion tunnels, one in either abutment were constructed in order to carry the river water. Two coffer dams enclosing the foundation area, were also constructed. Both the tunnels are 50 ft, in finished diameter, half a mile long and are lined with 3.6 ft thick heavily reinforced concrete. The work on these two tunnels was started in 1948 and was completed in 1953. The total expenditure incurred on them was approximately Rs. 3.6 crores.

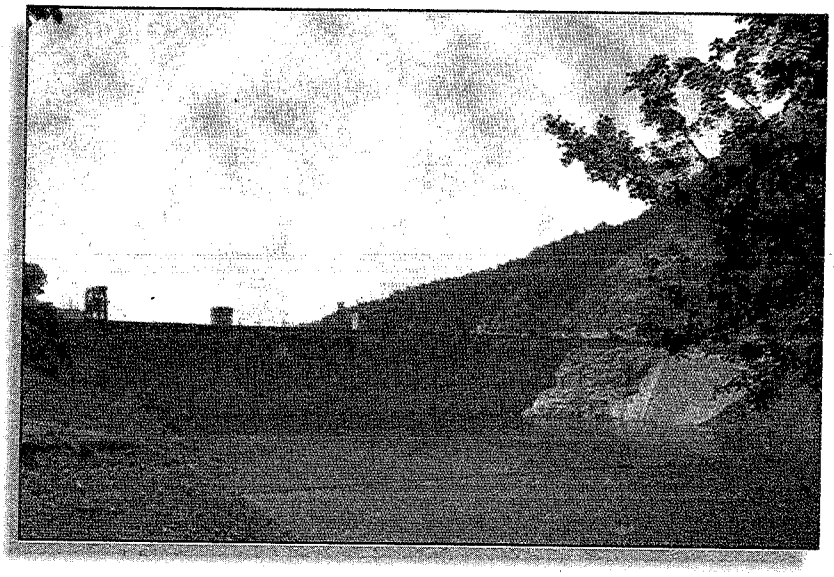


Fig. 17.4 (*a*) Upstream view of Bhakra Dam.

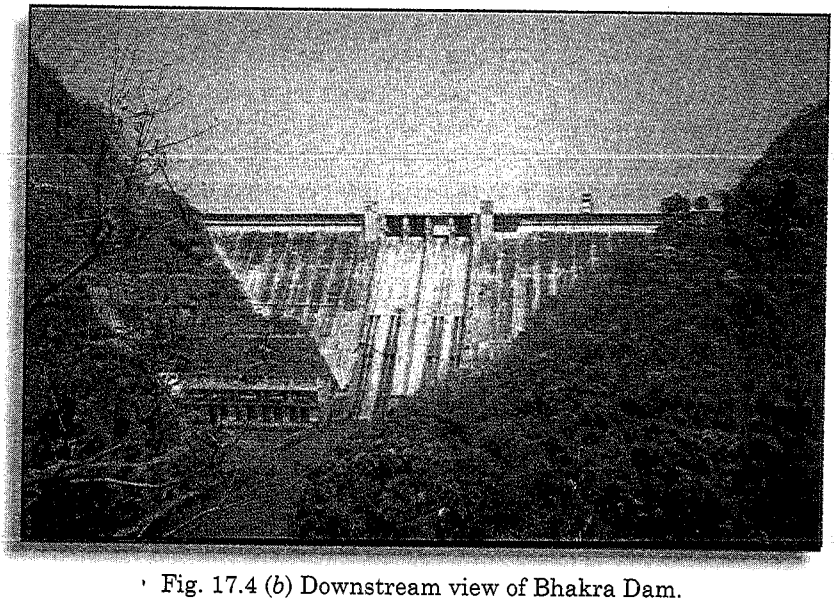


Fig. 17.4 (*b*) Downstream view of Bhakra Dam.

Fig. 17.4. Photoviews of Bhakra Dam.

After the construction of these tunnels, two earth (rolled) and rock coffer dams were constructed to enclose the operation area of the dam site. The construction of 215 ft high coffer dam began in 1956. The water was forced into the tunnels and taken downstream beyond the construction point. The 15 ft high downstream coffer dam did not let the water come back in the pit.

The foundations of these rolled fill earthen dams or dikes were carried down to 60ft and 70 ft below the river bed level. After the construction of coffer dams, the dewatering of the area was expeditiously completed and the excavation of the river bed started. The rock was blasted by an explosive. Desert-shovels were employed for loading the material in special trucks having capacity of 914 cubic yards. The material was carried and dumped at the dumping site. The total excavation was of the order of 700 million cubic yards. About one million cubic yard was excavated per day.

While the excavation was going on, a large constructional plant, capable of producing high quality concrete economically and efficiently was also installed. This required a $4\frac{1}{2}''$ long belt conveyor system, aggregate processing plant, cement handling plant, cooling plant, batching and mixing plants, high steel tristles, revolving cranes and other electricity operated cranes, etc. etc. It was an expensive, completely automised concrete plant. It was capable of handling about *600 tons* of concrete per hour.

Then, the actual concreting work of the dam started in November 1956. Huge electric cranes including 6 cantilever-cranes and 2 stiff-legged-cranes were employed for concreting of dam.

A very low heat cement was used in the construction of this dam. Because, when cement sets, it produces a large amount of heat which is liable to cause cracks in the structure. So in order to avoid this cracking, steel pipes were embedded in the structure and ice-cold water was circulated through them.

The first stage of the dam (390 ft.) was completed by 1959. Work in the 2nd stage was interrupted by a flood in the diversion tunnel that drowned ten workers and damaged the power house. The tunnel and plugging of the dam was completed in 1962.

Dimensions and other data about Bhakra Dam are given below:

Height of the dam	= 740′
Length at the bottom	= 325′
Length at the top	= 1200′
Breadth or thickness at the bottom	= 1320′
Breadth or thickness at the top	= 30′
Concrete required for the dam	= 55 lakh cubic yard
Electricity generated	= 12 lakh Kilowatt
Name of the reservoir formed	= Govind Sagar
Length of the reservoir	= 90 miles
Maximum depth	= 740′
Minimum depth	= 300′
Total irrigation	= 1 crore acre of area
Population of these villages	= 30, 000
Total storage capacity	= 8 million acre ft.
Catchment area	= 22, 000 sq. miles
Live storage	= 6.35 million acre ft.
No. of Inspection galleries	= 46

Power Houses. There are two power houses called (*i*) The Left Power House; and (ii) The Right Power House. The left power house is more important and was first constructed.

The left power house is a reinforced concrete structure. It is standing on a stone consisting partly of clay stone and partly of sand stone. The foundation chosen were adverse to the geological conditions required for a good dam site. The permeability of sand stone is high, and thus there was a possibility of leakage and danger of undermining of the foundations. It necessitated a heavily reinforced raft foundation. The left power house building is seven storeyed and its construction took about two to three years. It required about 635, 000 cubic yards of concrete, about one million sq.ft. of finishing work and 6, 000 tons of reinforced steel.

The machinery used in this power house is of cosmopolitan in nature. The turbines were supplied by Japan, the generators and transformers by U.K., and over-head cranes by Yogoslavia.

The local manufacture of 10 ft diameter penstocks, made out of 1.75″ thick steel plate was considered as a big engineering feat of the country in such a short period of independence. The left power house had costed nearly Rs. 10 crores. The work on right power house was also completed afterwards..

Problem of Wood Transport. Like all other perennial rivers of India, the River Satluj has been the cheapest means of transporting wood from the Himalayan forests down to the plains. But since the dam is very high above the river water level, it has become impossible to use this river for this purpose.

The problem is overcome by bringing logs of woods from Govind Sagar to Nangal railway station by means of an aerial rope-way. This is about $5\frac{1}{2}$ miles long. A wooden log loom has been put across Govind Sagar to obstruct timber. From there, the timber is taken by the inclined carriage way to the loading station on the upstream of the right side.

17.8.1. Nangal Dam. Nangal Dam is 95 ft high subsidiary dam, 8 miles downstream of Bhakra on Satluj. It falls within the jurisdiction of Punjab State in India. The length of the dam is 1, 000 ft. The object of this dam is to head up water of the river Satluj and then divert it into the canal off-taking from the left bank of the river. The canal is called Nangal Hydel Canal, and is a 40 miles long concrete lined canal.

This dam has 29 strong gates of span 30 ft each. An enormous tunnel called the Inspection gallery has been made in the river Satluj in the lower portion of this dam. This was the first tunnel constructed by the Government under Bhakra Project Scheme. In order to enter into this tunnel, one has to go 70 ft down. The tunnel goes across Satluj River.

Bhakra Nangal Project is something tremendous, stupendous, something which shakes up and thrills us when we see it. It marks the India's progress after her Independence. It is something which cannot be forgotten easily, if we see it once.

17.9. Nagarjuna Sagar Dam

The multipurpose Nagarjuna Sagar dam is located across Krishna River, near Nandikonda village in Nalgonda District (Andhra Pradesh). It is named after Buddhist Savant, Acharya Nagarjuna, who lived at the spot about 2,000 years ago to fulfil a mission.

This Dam irrigates in Guntur, Kurnool, Nellore, Nalgonda, Khamman and Krishna Districts. Its irrigation Potential is about 35 lakh acres of land and electrical potential is 1 lakh kilowatt hours of firm power (guaranteed-power generation) and 4.6 lakh kilowatt hours of seasonal power.

A photoview of this dam is shown in Fig. 17.5.

Salient Features of the Dam. The river gorge is blocked by a masonry dam 409 ft (124.6 m) high above the deepest foundation level and 4756 ft long. The full reservoir level is +590.0 ft above the mean sea level (M.S.L.). The most unique feature of this dam is the adoption of Stone masonry for its construction, deviating from the traditional concrete. The use of stone had resulted in a large saving and had created huge employment potential for a large labour force. Hence, Nagarjuna Sagar Project ranks first in the man-power utilisation among the modern gigantic projects of its own kind in the world. It was designed and executed entirely by Indian engineers. On either side of the masonry dam, earth dams have been constructed for a length of about two miles, the maximum height being 85 ft. The spillway crest has been installed with 26 Radial gates each of size 45′×44′. Other component-works of this mighty dam include 8 penstock pipes on the left side, three Power sluices and 9 irrigation sluices on the right side, two chute sluices and a diversion-cum-irrigation tunnel. Two canals-off-take on either side of the dam for irrigation. The expenditure on the project was of the order of Rs. 80 crores. The crest level of the dam is +605 ft and the crest level of spillway is +546 ft. Various details of the dam are given below:

(1) **Location.** Lat. = 16°34′ North, Long. = 71°19′ East, $1\frac{1}{2}$ mile downstream of Nandikonda village Miryalaguda Taluk, Nalgonda District, 90 miles from Hyderabad.

(2) **States Covered.** ANDHRA PRADESH

(3) **Hydrology**

(*a*) Water-shed area at dam site	... 83,087 sq. miles
(*b*) Maximuml flood discharge (observed)	... 11.7 lakh cumecs.

(4) **Reservoir**

(*a*)	Full Reservoir level	... +590′.0
(*b*)	Maximum water level	... +594′.0
(*c*)	Gross storage capacity	... 9.37 M.a.ft
(*d*)	Net (live) storage capacity	... 5.51 M.a.ft
(*e*)	No. of villages submerged	... 57
(*f*)	Population displaced	... 4,824 families

(5) **Masonry dam**

(*a*)	Total length	... 4,756 ft
(*b*)	Spillway length	... 1545 ft
(*c*)	Non-overflow length including power dam	... 3,211 ft
(*d*)	Height of dam (maximum)	... 409 ft
(*e*)	Base width (maximum)	... 320 ft
(*f*)	Top width	... 28 ft

(*g*)	Top level	... + 605′.0
(*h*)	Top of crest in spillway	... + 546′.0
(*i*)	Chute sluices 2 No. 10′ × 25′ provided in blocks 25 and 51 with sill level elevation	... + 450′.0
(*j*)	8 No. of 16′ diameter penstocks provided one in each in blocks 16 to 23 with central line in elevation	... + 405′.0
(*k*)	3 No. power sluices 15′ × 38′ provided (two in block 71 and one in block 72) with sill level at elevation	... + 479′.0
(*l*)	Radial Crest Gates 26 No. 45′ × 44′, with crest elevation	... + 546′.0
(*m*)	Thickness of spillway pipes	... + 15′
(*n*)	Nine No. sluices 10′ × 15′ are provided for right canal head regulator with sill at elevation	... + 489′.0

(6) **Quantities of Work**

(*a*)	Excavation for foundations	... + 42.07 M. cft
(*b*)	Volume of masonry and concrete	... 198 M. cft
(*c*)	Quantity of cement	... About 11 lakh tons
(*d*)	Quantity of steel	... About 60,000 tons

(7) **Earth Dams**

(*a*)	Length of Left Earth Dam	... 8,400 ft
(*b*)	Length of Right Earth Dam	... 2,800 ft
		11,200 ft
(*c*)	Maximum height above foundation level	... 85 ft
(*d*)	Top width	... 30 ft
(*e*)	Top level	... 610′
(*f*)	Excavation of foundation	... 9.2 M. cft
(*g*)	Earthwork for embankment	... 88 M. cft

(8) **Irrigation-cum-Diversion Tunnel (horse-shoe in Section)**

(*a*)	Length	... 2,590 ft
(*b*)	Diameter	... 27 ft

(9) **Power Plant**

(*a*)	Left side 8 No. of penstocks 16 ft. diameter to develop	... 3 lakh kW
(*b*)	Right side 3 power sluices of size 15′ × 38′ to develop	... 0.6 lakh kW

(10) **Man-Power** ... 50,000

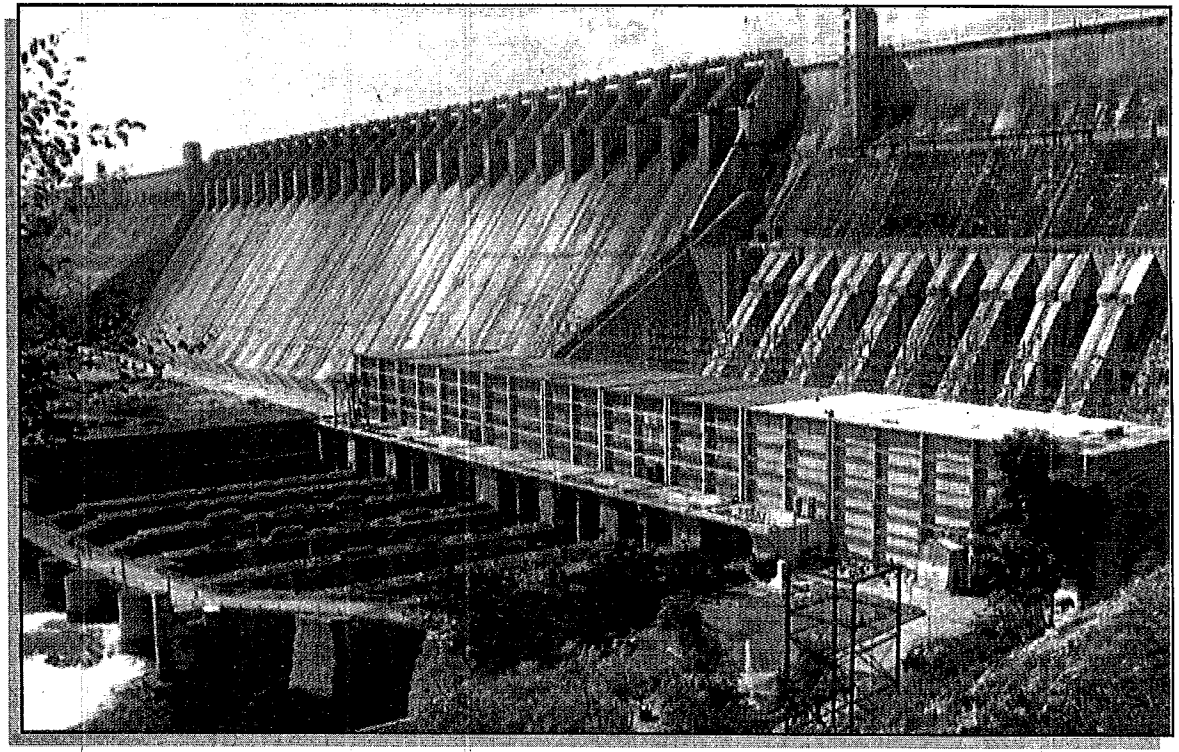


Fig. 17.5. Photoview of Nagarjuna Sagar Dam.

Organisation. In the earlier years of construction, a Control Board was the orverall incharge of the Nagarjun Sagar Project including technical, financial and administration aspects. The project was then located in the Andhra Pradesh and the Hyderabad States. After the reorganisation of States, the project came entirely in Andhra Pradesh State. The State Government assumed its full responsibility w.e.f. 1.8.1959. The Control Board became an advisory Body to the State Government, whose advice is accepted as a matter of convention.

The programme of construction of dam and canals was so adjusted that partial benefits started accruing even from 1967. The reservoir was able to deliver water to first crop of Krishna delta in time for raising substantial acreage in 2nd crop. Letting out waters in the two canals was inaugurated, by Smt. Indira Gandhi, the then Prime Minister of India, on 4.8.1967.

Rehabilitation. The man-made lake of Nagarjuna Sagar fully submerged lands of 52 villages and partially submerged that of 5 villages.

About 4,900 families were displaced and rehabilitated in 24 Rehabilitation centres. Good facilities, liberal compensation and other amenities were provided to those who had to be uprouted.

Nagarjunakonda Excavations. As the Nagarjuna Sagar reservoir was to completely submerge the famous relics of Nagarjuna Konda, which was the seat of Ikshwaku Kings and one of the principal centres of Mahayana System of Buddhism, the whole area was excavated by the Archeological Department of the Governmernt of India. The more important of the relics were located in a museum constructed for the purpose, on the top of an adjoining hill. Practically, all the relics had been unearthed and shifted to the museum. Expenditure to the extent of 12 lakh rupees was debited to the Project and the balance was met by the Archeological department.

PROBLEMS

1. (*a*) What is meant by a "dam and a reservoir" ? What are the different meterials that are commonly used for dam construction and what are their comparative advantages and disadvantages ?

(*b*) Discuss the geological and tropological features which affect the selection of the type of dam.

2. (*a*) What are 'arch' and 'buttress' dams ? Illustrate with sketches and mention site conditions favourable for construction of such dams.

(*b*) Discuss the factors which are considered in the selection of the site for a proposed dam. It is assumed that the type of the dam has already been selected for the project.

3. (*a*) What useful purpose is served by a dam ? What are the illeffects of dam construction ?

(*b*) How do you classify dams according to :

(*i*) their use ;

(*ii*) their hydraulic designs ;

(*iii*) their materials of construction.

(*c*) Discuss the various factors which govern the selection of a particular type of dam for a particular project.

4. "Dams are the sources of sorrow and grief". Debate the above statement giving points in favour as well as against it.

5. Narrate briefly the story of construction of any major dam of India.

18

Reservoirs and Planning for Dam Reservoirs

18.1. Definition and Types

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, generally called a *dam reservoir* or an *impounding reservoir* or *a river reservoir.*

The quality of water stored in such a reservoir is not much different from that of a natural lake. The water so stored in a given reservoir during rainy season can be easily used almost throughout the year, till the time of arrival of the next rainy season, to refill the emptying reservoir again.

Depending upon the purpose served by a given reservoir, the reservoirs may be broadly divided into the following three types.

(1) *Storage or Conservation reservoirs* ;

(2) *Flood Control reservoirs* ; and

(3) *Multipurpose reservoirs.*

The fourth type of a reservoir is a simple storage tank constructed within a city water supply system, and is called a *Distribution reservoir* ; and such a reservoir is evidently not a river reservoir, but is a simple storage tank.

18.1.1. Storage or Conservation Reservoirs. A city water supply, irrigation water supply, or a hydroelectric project drawing water directly from a river or a stream may fail to satisfy the consumers demands during extremely low flows; while during high flows, it may become difficult to carry out their operations due to devastating floods. *A storage or a conservation reservoir can retain such excess supplies during periods of peak flows, and can release them gradually during low flows as and when the need arises.*

Incidentally, in addition to conserving water for later use, the storage of flood waters may also reduce flood damage below the reservoir. Hence, a reservoir can be used for controlling floods either solely or in addition to other purposes. In the former case, it is known as a 'Flood Control Reservoir' or a 'Single Purpose Flood Control Reservoir'; and in the latter case, it is called a 'Multipurpose Reservoir'.

18.1.2. Flood Control Reservoirs. A flood control reservoir, generally called a flood-mitigation reservoir, stores a portion of the flood flows in such a way as to minimise the flood peaks at the areas to be protected downstream. To accomplish this, the entire inflow entering the reservoir is discharged till the outflow reaches the safe capacity of the channel downstream. The inflow in excess of this rate is stored in the

reservoir, which is then gradually released, so as to recover the storage capacity for the next flood.

The flood peaks at the downstream of the reservoir are thus reduced by an amount *AB*, as shown in Fig. 18.1. A flood control reservoir differs from a conservation reservoir only in its need for a large sluiceway capacity to permit rapid drawdown before or after a flood.

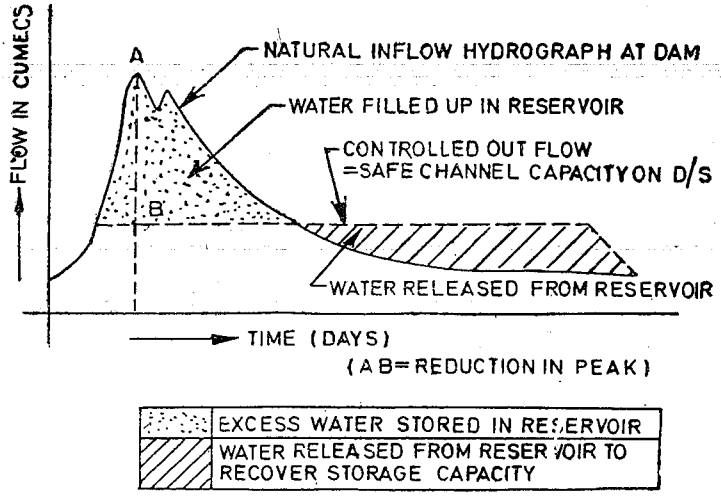


Fig. 18.1.

Types of flood control reservoirs. There are two basic types of flood-mitigation reservoirs; *i.e.*

(*i*) *Storage reservoirs or Detention basins* ; and

(*ii*) *Retarding basins or Retarding reservoirs.*

A reservoir having gates and valves installation at its spillway and at its sluice outlets is known as a storage reservoir; while on the other hand, a reservoir with uncontrolled and ungated outlets is known as a **retarding basin.**

Functioning and Advantages of a Retarding Basin

A retarding basin is usually provided with an uncontrolled spillway and an uncontrolled orifice type sluiceways. The automatic regulation of outflow, depending upon the availability of water, takes place from such a reservoir. The maximum discharging capacity of such a reservoir should be equal to the maximum safe carrying capacity of the channel downstream. As floods occur, the reservoir gets filled, and discharges through sluiceways. As the reservoir elevation increases, the outflow discharge increases. The water level goes on rising until the flood has subsided, and the inflow becomes equal to or less than the outflow. After this, the water gets automatically withdrawn from the reservoir until the stored water is completely discharged. The advantages of a retarding basin over a gate controlled detention basin are:

(*i*) Cost of the gate installation is saved.

(*ii*) There are no gates and hence, the possibility of human error and negligence in their operation is eliminated.

(*iii*) Since such a reservoir is not always filled, much of the land below the maximum reservoir level will be submerged only temporarily and occasionally, and can be successfully used for agriculture, although no permanent habitation can be allowed on this land.

Functioning and Advantages of a Storage Reservoir

A storage reservoir with gated spillway and gated sluiceways, provides more flexibility of operation, and thus gives us better control and increased usefulness of the reservoir. Storage reservoirs are, therefore, preferred on large rivers, which require better control; while retarding basins are preferred on small rivers. In storage reservoirs, the flood crest downstream, can be better controlled and regulated properly, so as not to cause their coincidence. This is the biggest advantage of such a reservoir and outweighs its disadvantages of being costly and involving risk of human error in installation and operation of gates.

18.1.3. Multipurpose Reservoirs. A reservoir planned and constructed to serve not only one purpose but various purposes together is called a multipurpose reservoir. Reservoir, designed for one purpose, incidentally serving other purposes, shall not be called a multipurpose reservoir, but will be called so, only if designed to serve those purposes also in addition to its main purpose. Hence, a reservoir designed to protect the downstream areas from floods and also to conserve water for water supply, irrigation, industrial needs, hydroelectric purposes, etc. shall be called a multipurpose reservoir. Bhakra dam and Nagarjun Sagar dam are the important multipupose projects of India.

18.1.4. Distribution Reservoirs. A distribution reservoir is a small storage reservoir constructed within a city water supply system. Such a reservoir can be filled by pumping water at a certain rate and can be used to supply water even at rates higher than the inflow rate during periods of maximum demands (called critical periods of demand). Such reservoirs are, therefore, helpful in permitting the pumps or the water treatment plants to work at a uniform rate, and they store water during the hours of no demand or less demand, and supply water from their 'storage' during the critical periods of maximum demand.

In this chapter, we shall however, confine ourselves to the river reservoirs only.

18.2. Capacity-Elevation and Area-Elevation Curves of a Reservoir Site

Whatever be the size or use of a reservoir, the main function of a reservoir is to store water and thus to stabilize the flow of water. Therefore, the most important physical characteristic of a reservoir is nothing but its *storage capacity*. The capacity of reservoirs on dam sites, is determined from the contour maps of the area. A topographic survey of the dam site is carried out, and a contour map such as shown in Fig. 18.2 is drawn. The area enclosed within each contour can be measured with a planimeter.

In fact, the general practice adopted for capacity computations is to actually survey the site contours only at vertical distances of 5 m or so. The areas of the intervening contours at smaller intervals of say 0.5 m or so, are then interpolated by taking the square root of the surveyed contours, and to assume that the square root of the interpolated ones, vary in exact proportion to their vertical distance apart. For example, suppose the area of the reservoir at 200 m contour is A_1 hectares, and that at 205 m contour is A_2 hectares; then

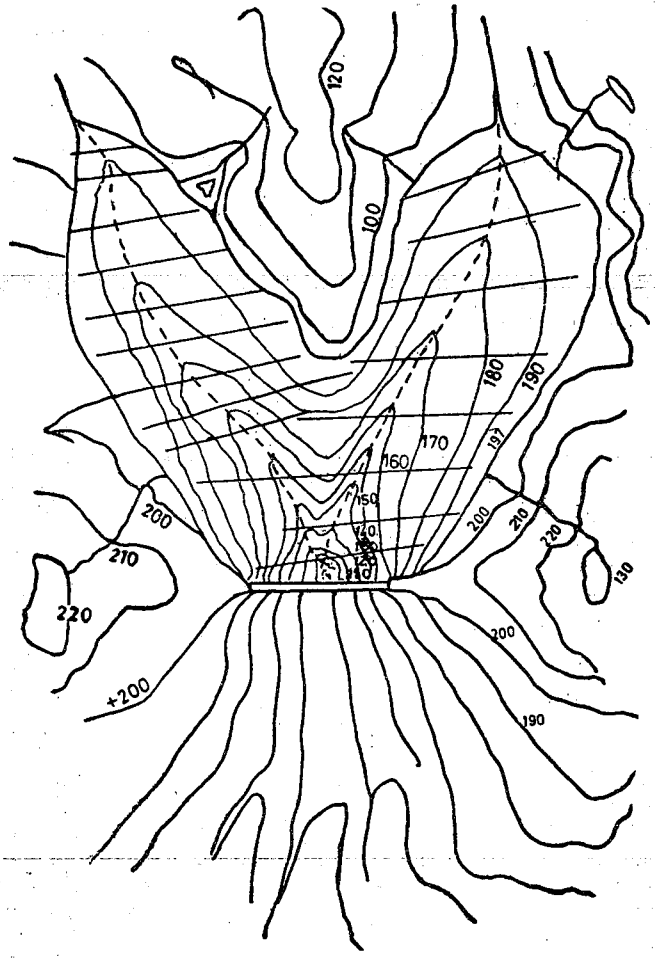


Fig. 18.2. A typical contour map of a dam site.

The area at 200.5 m contour is

$$= \left[\sqrt{A_1} + \frac{200.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{0.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad \text{...[18.1 (}a\text{)]}$$

Similarly, the area at 201.0 m contour

$$= \left[\sqrt{A_1} + \frac{201.0 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{1.0}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad \text{...[18.1 (}b\text{)]}$$

Similarly, the area at 204.5 m contour

$$= \left[\sqrt{A_1} + \frac{204.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{4.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad ...[18.1\ (c)]$$

In this way, the areas can be computed at sufficiently low contour intervals (0.5 m), which can be used to determine the incremental volumes (ΔS) stored between two successive contours, by using the simple average method, *i.e.* by multiplying the average of the two areas at the two elevations, by the elevation difference (Δh). The summation of these incremental volumes below any elevation, is the storage volume below that level.

Instead of using the simple average formula, *i.e.*

$$\Delta S = \frac{a_1 + a_2}{2}(\Delta h), \text{ sometimes the formula}$$

$$\Delta S = \frac{\Delta h}{3}\left[a_1 + a_2 + \sqrt{a_1 a_2}\right] \text{ can also be used, where}$$

a_1, a_2, ...represent the areas at 0.5 m contour interval.

Prismoidal formula can also be used preferably where three consecutive sections at equal height are taken. According to this,

$$\Delta S = \text{Storage} = \frac{\Delta h}{6}\left[A_1 + 4A_2 + A_3\right] \quad ...(18.2)$$

where A_1, A_2 and A_3 are the areas of succeeding contours, and Δh is the vertical distance between two alternate contours.

It thus becomes evident that the areas at different elevations (contours), as well as the storage at different elevations, can be mathematically worked out, and both plotted on a graph paper, to obtain **Area-Elevation curve, and Storage-Elevation curve** (*Capacity-Elevation curve*), respectively, as shown in Fig. 18.3.

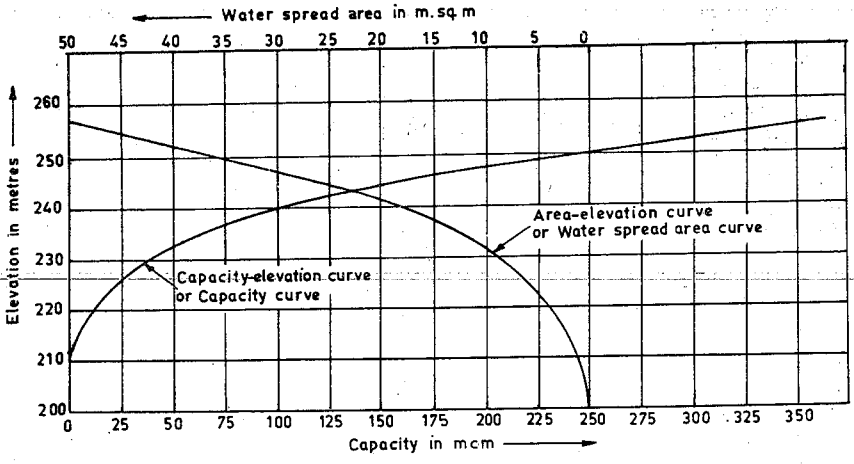


Fig. 18.3. Area elevation curve and Capacity elevation curve.

Another method for computing capacity may be the use of integration technique. Infact, the area-elevation curve, when integrated, will yield nothing but the capacity-elevation curve. Hence the surveyed areas at large-intervals, may be plotted on a simple graph paper, and a smooth curve, *i.e.* area-elevation curve, is first of all drawn. The equation of this curve is now obtained by statistical methods, which can be integrated to obtain the equation of the capacity-elevation curve, as follows:

The equation of the area-elevation curve, will generally be of the form:

$$A = \alpha + \beta \,.\, h + \gamma \,.\, h^2 + \ldots \eta \,.\, h^{n-1} \qquad \ldots(18.3)$$

where A represents the area at any elevation h; and
$\alpha, \beta, \gamma \ldots \eta$ are all constants.

This equation can be determined and then integrated to obtain the storage (capacity), as explained below:

Let y be the height of the water surface in the reservoir above any assumed datum, over which the storage/capacity is to be worked out.

Let A_y represents symbolically the area of the contour at this height. Then, assume that the *equation of area-elevation curve* is given by :

$$A_y = \alpha + \beta \,.\, y + \gamma \,.\, y^2 + \ldots \eta \,.\, y^{n-1} \qquad \ldots(18.3\ a)$$

where $\alpha, \beta, \gamma, \ldots, \eta$ are all constants.

From the actual survey, or from the points falling on the area-elevation curve, the area of any required number of contours (n) are known.

Let them be A_0, A_1, A_2.....corresponding to the known level values, *i.e.*, heights $0, y_1, y_2, \ldots$ above the datum.

Substituting these values in equation (18.3 *a*), we get

$$A_0 = \alpha$$
$$A_1 = \alpha + \beta \,.\, y_1 + \gamma \,.\, y_1^2 + \ldots \eta \,.\, y_1^{n-1}$$
$$A_2 = \alpha + \beta \,.\, y_2 + \gamma \,.\, y_2^2 + \ldots \eta \,.\, y_2^{n-1}$$
$$A_3 = \alpha + \beta \,.\, y_3 + \gamma \,.\, y_3^2 + \ldots \eta \,.\, y_3^{n-1}$$

Thus, we get n simultaneous equations to determine n number of constants $(\alpha, \beta, \gamma, \ldots, \eta)$ Hence, the equation of the area-elevation curve, *i.e.* $A_y = \alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1}$ becomes defined, with $\alpha, \beta, \gamma, \ldots \eta$ all known.

This equation can now be integrated between the limits 0 to y,

$$\int_{y=0}^{y=y} A_y \cdot dy = \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

or S_y = storage (capacity) between 0 to y

$$= \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

$$= \left[\alpha.y + \beta.\frac{y^2}{2} + \gamma.\frac{y^3}{3} + \ldots \eta.\frac{y^n}{n} \right] + K$$

where K is a constant, which obviously is the reservoir capacity at datum.

The use of this method can be best understood by solving a numerical example.

Example 18.1. *A contour survey of a reservoir site gives the following data:*

Contour value	*Area*
At 200 m contour	*6.0 hectares*
At 210 m contour	*18.1 hectares*
At 220 m contour	*34.0 hectares*

The capacity of the reservoir upto 200 m elevation is found to be 14.1 ha. m. Determine the general equation for the area-elevation curve and capacity-elevation curve. Also determine the reservoir capacity at RL 225 m.

Solution. Use equation (18.3 *a*), wherein y is the height above *RL* 200 m, as

$$A_y = \alpha + \beta \,.\, y + \gamma \,.\, y^2$$

Now substituting the given values, we have

$$A_0 = 6.0 \text{ hectares}$$

$$\therefore \quad 6.0 = \alpha + \beta\,(0) + \gamma\,(0) = \alpha$$

or $$\alpha = \mathbf{6.0} \qquad ...(i)$$

Also $A_1 = 18.1$ hectares, at $y_1 = 10$ m (given)

$$\therefore \quad 18.1 = \alpha + \beta \,.\, (10) + \gamma \,.\, (10)^2$$

or $$18.1 = 6.0 + 10\beta + 100\gamma$$

or $$12.1 = 10\beta + 100\gamma$$

or $$\beta + 10\gamma = 1.21 \qquad ...(ii)$$

Also $A_2 = 34.0$ hectares at $y_2 = 20$ m (given)

$$\therefore \quad 34.0 = \alpha + \beta\,(20) + \gamma\,(20)^2 = 6.0 + 20\beta + 400\gamma$$

or $$20\beta + 400\gamma = 28.0$$

$$\beta + 20\gamma = 1.4 \qquad ...(iii)$$

Solving (*ii*) and (*iii*), we get

$$\beta + 10\gamma = 1.21$$

$$\beta + 20\gamma = 1.40$$

or $$10\gamma = 0.19 \quad \text{or} \quad \gamma = \mathbf{0.019}$$

$$\therefore \quad \beta + 0.19 = 1.21 \quad \text{or} \quad \beta = \mathbf{1.02}$$

Hence, the equation of area-elevation curve is given by

$$A_y = 6.0 + 1.02y + 0.019y^2 \quad \textbf{Ans.}$$

Integrating this equation, we get

$$S_y = 6.0y + 1.02\frac{y^2}{2} + 0.019\frac{y^3}{3} + K$$

The constant K is obviously the reservoir capacity (storage) upto *RL* 200.0 m, which is given to be 14.1 ha. m.

$$\therefore \quad S_y = 6.0y + 0.51y^2 + 0.0063y^3 + 14.1$$

Hence, the capacity-elevation curve is given by

$$S_y = 0.0063y^3 + 0.51y^2 + 6.0y + 14.1 \quad \textbf{Ans}$$

To determine the capacity at *RL* 225 m, we have to substitute $y = 225 - 200 = 25$ m in the above eqn. to obtain the requisite capacity in ha.m., *i.e.*

Required capacity at *RL* 225 m

$$= 0.0063\,(25)^3 + 0.51\,(25)^2 + 6.0 \times 25 + 14.1$$
$$= 98.96 + 318.75 + 150.0 + 14.1 = \mathbf{581.81\,ha.m.} \quad \textbf{Ans.}$$

Conclusions. It can thus be seen that the capacity of a reservoir by this method can be determined by surveying only a few contours. It is also found that the method does not give more than 3% error, when it is cross-checked with the capacity worked out by surveying large number of contours. This error is not considered much, in the light of the fact that the areas of contours are themselves not very precise figures.

Sometimes, storage capacity may be expressed as a single term function of y, such as

$$S_y = K.y^n \qquad \text{where } K \text{ and } n \text{ are constants.}$$

Infact, in practical life, no one bothers about the equations, and only curves, as shown in Fig. 18.3 are drawn. The required capacity at any elevation is read out, from such a curve.

It may also be pointed out here that the best capacity curve for a reservoir is the one in which the rise to the straight line is the quickest. It results from a cup-shaped catchment, with gentle longitudinal slope. Fig. 18.4 shows the good and bad capacity curves.

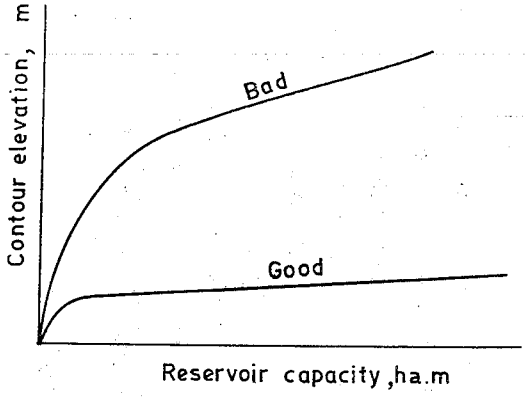


Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

These zones are defined w.r. to Fig. 18.5.

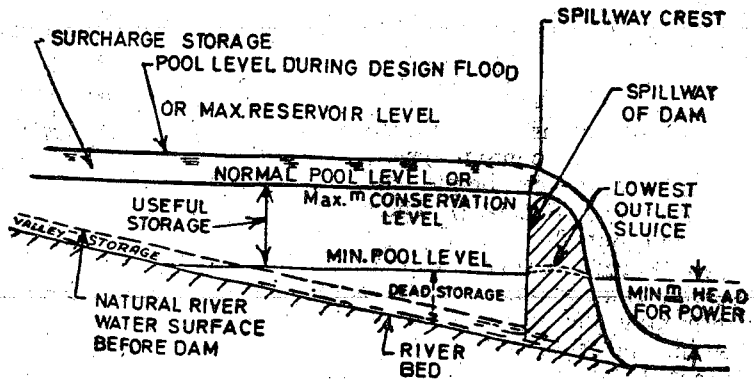


Fig. 18.5. Storage zones of a reservoir.

18.3.1. Normal Pool Level or Maximum Conservation Level. It is the maximum elevation to which the reservoir water surface will rise during normal operating conditions. (See Fig. 18.5). It is equivalent to the elevation of the spillway crest or the top of the spillway gates, for most of the cases.

18.3.2. Minimum Pool Level. The lowest water surface elevation, which has to be kept under normal operating conditions in a reservoir, is called the minimum pool level (See Fig. 18.5). This level may be fixed by the elevation of the lowest outlet in the dam or may be guided by the minimum head required for efficient functioning of turbines.

18.3.3. Useful and Dead Storage. *The volume of water stored in a reservoir between the minimum pool and normal pool levels is called the useful storage.* Water stored in the reservoir below the minimum pool level is known as the *Dead Storage,* and it is not of much use in the operation of the reservoirs. The useful storage may be subdivided into conservation storage and flood-mitigation storage, in a multipurpose reservoir.

18.3.4. Maximum Pool Level or Full Reservoir Level. During high floods, water is discharged over the spillway, but will cause the water level to rise in the reservoir above the normal pool level. The maximum level to which the water rises during the worst design flood is known as the *maximum pool level.*

18.3.5. Surcharge Storage. The volume of water stored between the normal pool level and the maximum pool level is called *surcharge storage.* Surcharge storage is an uncontrolled storage, in the sense that it exists only till the flood is in progress and cannot be retained for later use.

18.3.6. Bank Storage. When the reservoir is filled up, certain amount of water seeps into the permeable reservoir banks. This water comes out as soon as the reservoir gets depleted. This volume of water is known as the *bank storage,* and may amount to several percent of the reservoir volume depending upon the geological formations. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation capacity curve of the reservoir.

18.3.7. Valley Storage. Even before a dam is constructed, certain variable amount of water is stored in the stream channel, called *valley storage.* After the reservoir is formed, the storage increases, and the actual net increase in the storage is equal to the storage capacity of the reservoir minus the natural valley storage. The valley storage thus reduces the effective storage capacity of a reservoir. It is not of much importance in conservation reservoirs, but the available storage for flood mitigation is reduced, as given by the following relation:

Effective storage for flood mitigation
= Useful Storage + Surcharge Storage – Valley Storage corresponding to the rate of inflow in the reservoir.

DESIGNING RESERVOIR CAPACITY

18.4. Catchment Yield and Reservoir Yield

Long range runoff from a catchment is known as the yield of the catchment. Generally, a period of one year is considered for determining the yield value. *The total yearly runoff, expressed as the volume of water entering/passing the outlet point of the catchment, is thus known as the catchment yield, and is expressed in M m^3 or M.ha.m.*

The annual yield of the catchment upto the site of a reservoir, located at the given point along a river, will thus indicate the quantum of water that will annually enter the reservoir, and will thus help in designing the capacity of the reservoir. This will also help to fix the outflows from the reservoir, since the outflows are dependent upon the inflows and the reservoir losses.

The amount of water that can be drawn from a reservoir, in any specified time interval, called the *reservoir yield,* naturally depends upon the inflow into the reservoir and the reservoir losses, consisting of reservoir leakage and reservoir evaporation.

The annual inflow to the reservoir, i.e. the *catchment yield,* is represented by the *mass curve of inflow;* whereas, the outflow from the reservoir, called the *reservoir yield,* is represented by the *mass demand line* or *the mass curve of outflow.* Both these curves decide the reservoir capacity, provided the reservoir losses are ignored or separately accounted.

The inflows to the reservoir are however, quite susceptible of variation in different years, and may therefore vary throughout the prospective life of the reservoir. *The past available data of rainfall or runoff in the catchment is therefore used to work out the optimum value of the catchment yield.* Say for example, in the past available records of say 35 years, the minimum yield from the catchment in the worst rainfall year may be as low as say 100 M.ha.m; whereas, the maximum yield in the best rainfall year may be as high as say 200 M.ha.m. The question which then arises would be as to whether the reservoir capacity should correspond to 100 M. ha.m yield or 200 M. ha.m yield. If the reservoir capacity is provided corresponding to 100 M. ha.m yield, then eventually the reservoir will be filled up every year with a dependability of 100%; but if the capacity is provided corresponding to 200 M.ha.m yield, then eventually the reservoir will be filled up only in the best rainfall year (*i.e.* once in 35 years) with a dependability of about $\frac{1}{35} \times 100 \approx 3\%$.

In order to obtain a sweet agreement, a via media is generally adopted and an intermediate *dependability percentage value* (p), such as 50% to 75%,* may be used to compute the **dependable yield** or **the design yield.** The yield which corresponds to the worst or the most critical year on record is however, called the **firm yield** or **the safe yield.** Water available in excess of the firm yield during years of higher inflows, is designated as **the secondary yield.** Hydropower may be developed from such secondary water, and sold to the industries *'on and when available basis'.* The power commitments to domestic consumers must, however, be based on the firm basis, and should not exceed the power which can be produced with the firm yield, unless thermal power is also available to support the hydroelectric power.

The arithmetic average of the firm yield and the secondary yield is called the **average yield.**

18.4.1. Computing the Design or the Dependable Catchment Yield. The dependable yield, corresponding to a given dependability percentage p, is determined from the past available data of the last 35 years or so. The yearly rainfall data of the reservoir catchment is generally used for this purpose, since such long runoff data is rarely available . The rainfall data of the past years is therefore used to work out the dependable rainfall value corresponding to the given dependability percentage p. This dependable rainfall value is then converted into the dependable runoff value by using the available empirical formulas connecting the yearly rainfall with the yearly runoff.

It is, however, an adopted practice in Irrigation Departments to plan the reservoir project by computing the dependable yield from the rainfall data, but to start river gauging as soon as the site for the reservoir is decided, and then correlate the rainfall-runoff observations to verify the correctness of the assumed empirical relation between the rainfall and the runoff. Sometimes, on the basis of such observations, the initially assumed yield value may have to be revised.

* This percentage will depend upon the risk which can be absorbed for the proposed use of water. Say for example, city water supply projects can absorb lesser risk as compared to the irrigation projects, and hence should consider higher dependability percentage value.

The procedure which is adopted to compute the dependable rainfall value for a given dependability percentage p is explained below, and has been further used in solving example 18.2.

(*i*) The available rainfall data of the past N years is first of all arranged in the descending order of magnitude.

(*ii*) The order number m, given by the equation

$$m = N \cdot \frac{p}{100} \qquad ...(18.4)$$

is then computed, and rainfall value corresponding to this order number in the tabulated data will represent the required dependable rainfall value.

(*iii*) If the computed value of m is a fraction, then the arithmetic mean of the rainfall values corresponding to whole number m values above and below this fraction value is taken as the **dependable rainfall** value.

This method of computing the rainfall value of the given dependability (such as 50%, 75%, etc.) will become more clear when we solve example 18.2. The rainfall value of given dependability can finally be converted into the dependable yield, by using empirical methods, discussed in article 18.4.1.1.

The dependable yield of a given dependability (p per cent) can also be determined from the stream gauging, by computing the annual yields, from the observed discharges, and arranging the annual yield values in descending order to determine order numbers (instead of rainfall) as exhibited in solved example 18.3.

Example 18.2. *The yearly rainfall data for the catchment of a proposed reservoir site for 35 years is given in Table 18.1. Compute from this data, the values of dependable rainfalls for 60% and 75% dependability percentage.*

Table 18.1

Year	*Rainfall in cm*	*Year*	*Rainfall in cm*
1956	98	1974	88
1957	100	1975	94
1958	101	1976	107
1959	99	1977	110
1960	85	1978	208
1961	112	1979	114
1962	116	1980	104
1963	78	1981	120
1964	160	1982	108
1965	66	1983	102
1966	184	1984	80
1967	90	1985	109
1968	76	1986	122
1969	118	1987	115
1970	86	1988	140
1971	92	1989	138
1972	96	1990	60
1973	93		

Solution. The given data of table 18.1 is arranged in descending order, mentioning serial number (order number m) in front of each, as shown in Table 18.2.

Table 18.2

S.No. i.e order number (m)	*Rainfall in desecending order in cm*	*S. No. i.e. order number (m)*	*Rainfall in descending order in cm*
1	208	19	101
2	184	20	100
3	160	21	99
4	140	22	98
5	138	23	96
6	122	24	94
7	120	25	93
8	118	26	92
9	116	27	90
10	115	28	88
11	114	29	86
12	112	30	85
13	110	31	80
14	109	32	78
15	108	33	76
16	107	34	66
17	104	35	60
18	102		

Now, using equation (18.4), we compute the order number (m) for the given dependability percentage $p = 60\%$, as:

$$m = N \cdot \frac{p}{100} = 35 \times \frac{60}{100} = 21$$

The rainfall value tabulated above in table 18.2 at order no.21 is 99 cm; and hence the required dependable rainfall = 99cm. $\therefore P_{60\%} = \mathbf{99\ cm}$ Ans.

Similarly, for dependability $p = 75\%$, we calculate the order no. (m)

$$= N \cdot \frac{p}{100} = 35 \times \frac{75}{100} = 26.25.$$

Since reqd. order no. is not an integer, the mean value of rainfall corresponding to order no. 26 and 27 will be taken as the value of p of 75% dependability.

$$\therefore \qquad P_{75\%} = \frac{92 + 90}{2} = 91 \text{ cm} \quad \textbf{Ans.}$$

Example 18.3. *The daily flows in a river for three consecutive years are given in the table by class interval alongwith the number of days the flow belonged to this class. What are the 50% and 75% dependable flows (annual and daily) for the river ?*

Table 18.3

		Year 1981	*Year 1982*	*Year 1983*
S. No.	*Daily mean discharge m^3/s (range)*	*No. of days the flow belonged to the given range (class interval)*	*No. of days the flow equalled the class range of col (2)*	*No. of days the flow equalled the range given in col (2)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1	*100-90.1*	*0*	*6*	*10*
2	*90-80.1*	*16*	*19*	*16*
3	*80-70.1*	*27*	*25*	*38*
4	*70-60.1*	*21*	*60*	*67*
5	*60-50.1*	*43*	*51*	*58*
6	*50-40.1*	*59*	*38*	*38*
7	*40-30.1*	*64*	*29*	*70*
8	*30-20.1*	*22*	*48*	*29*
9	*20-10.1*	*59*	*63*	*26*
10	*10-negligible*	*54*	*26*	*13*

(Engg. Services, 2002)

Solution. With the given daily flows, we will calculate the annual yields for the given 3 years of records, as computed in Table 18.4. Since daily discharge figs. are given in range (interval), mid value of range (interval) can be assumed to be the flow discharge, which when multiplied by no. of days, will give yield during those days. Summation of yield values over 365 days will give annual yield, as shown in Table 18.4, which otherwise is self explanatory.

Table 18.4

			Year 1981		Year 1982		Year 1983	
S. No.	Daily mean discharge (range) (m^3/s) in descending order	Mean value of daily discharge range	No. of days for which the flow equalled to that given in col (3)	Yield (Mm^3) = (3) × (4) × $\frac{86400}{10^6}$ [(3) × (4) × 0.0864]	No. of days for which flow equalled to that of col (3)	Yield (Mm^3) = 0.0864 × col (3) × col (6)	No. of days for which flow equaled to that of col (3)	Yield ($M\,m^3$) = 0.0864 × col (3) × col (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	100-90.1	95.05	0	0	6	49.25	10	82.08
2	90-80.1	85.05	16	117.50	19	139.54	16	117.50
3	80-70.1	75.05	27	174.96	25	162.00	38	246.24
4	70-60.1	65.05	21	117.94	60	336.96	67	376.27
5	60-50.1	55.05	43	204.34	51	242.35	58	275.62
6	50-40.1	45.05	59	229.39	38	147.77	38	147.74
7	40-30.1	35.05	64	193.54	29	87.70	70	211.68
8	30-20.1	25.05	22	47.52	48	103.68	29	62.64
9	20-10.1	15.05	59	74.34	63	81.65	26	33.70
10	10-Negligible	5.05	54	23.33	26	11.23	13	5.62
	Σ		365	1182.86	365	1361.98	365	1559.09

The river yield for 3 year is thus, calculated by summation of col (5), (7) & (9). Let these yields be arranged in descending order.

Year	Annual yield ($M\,m^3$)	Order No. (m)
1983	1559.09	1
1982	1361.98	2
1981	1182.86	3
		$N = 3$

Order No. (m) for p% dependability

$$= \frac{p}{100} \times N \qquad \text{...(Eqn. 18.4)}$$

(1) Order No for 75% dependability $= \frac{75}{100} \times 3 = 2.25$.

Since order No. of 2.25 is not an integer, the mean value of yield corresponding to S. NO. 2 and 3 will be taken as yield for 75% dependability, given as

$$= \left[\frac{1361.98 + 1182.86}{2}\right] = 1272.42\,M\,m^3$$

Hence, Annual flow with 75% dependability = **1272.42 M m³** **Ans.**

∴ Daily flow (discharge) with 75% dependability

$$= \frac{1272.42 \times 10^6}{365 \times 86400}\ m^3/s = \mathbf{40.34^3\ m^3/s} \quad \textbf{Ans.}$$

(*ii*) Order No. for 50% dependability $= \frac{p}{100} \times N = \frac{50}{100} \times 3 = 1.5$

Since order No. of 1.5 is not an integer, the mean value of yield corresponding to order No. 1 and 2 shall be taken as the yield of 50%. Dependability, given as :

$$= \frac{1559.09 + 1361.98}{2} = 1460.54\, M\, m^3$$

Hence, Annual flow with 50% dependability = **1460.54 M m^3** **Ans.**

∴ Daily flow (discharge) with 50% dependability

$$= \frac{1460.54 \times 10^6}{365 \times 86400}\, m^3/s = \mathbf{46.31\ m^3/s} \quad \textbf{Ans.}$$

18.4.1.1. Converting the dependable rainfall value into the dependable yield value. Certain empirical relations are available for converting the yearly rainfall value for the given catchment into the yearly runoff value expected from that catchment. Some of these formulas are:

1. *Binnie's percentages;*
2. *Strange's tables;*
3. *Barlow's tables;*
4. *Lacey's formula;*
5. *Inglis formula; and*
6. *Khosla's formula.*

These empirical relations are briefly discussed below:

(1) **Binnie's percentages.** The first effort ever made in India to connect the long range rainfall and the runoff (yield), was from Sir Alexander Binnie. He made observations *on two rivers in the central provinces,* and worked out certain percentages to connect the monthly rainfall with the monthly yield for the entire monsoon period from June to October. These percentages have been further adjusted by Mr. Garret, and are given in table 18.5. From the values given in table 18.5, *the total monsoon rainfall Vs percentage of rainfall that becomes runoff* are plotted in Fig. 18.6(*a*).

Table 18.5. Binnie's Monsoon Yield Percentages (as adjusted by Garret)

Monsoon Rainfall (P) in cm	*Monsoon Yield percentage*	*Monsoon Rainfall (P) in cm*	*Monsoon Yield percentage*
25	7.0	107.5	39.5
30	9.0	110.0	40.3
35	11.0	112.5	41.0
40	13.0	115.0	41.7
45	15.0	117.5	42.4
50	17.0	120.0	43.1
55	19.0	122.5	43.7
60	21.0	125.0	44.4
65	23.0	127.5	45.0
70	25.0	130.0	45.6
75	27.0	132.5	46.2
80	29.0	135.0	46.8
85	31.0	137.5	47.3
90	33.0	140.0	47.9
95	35.0	142.5	48.4
100	37.0	145.0	49.0
102.5	37.9	147.5	49.5
105.0	38.7	150.0	50.0

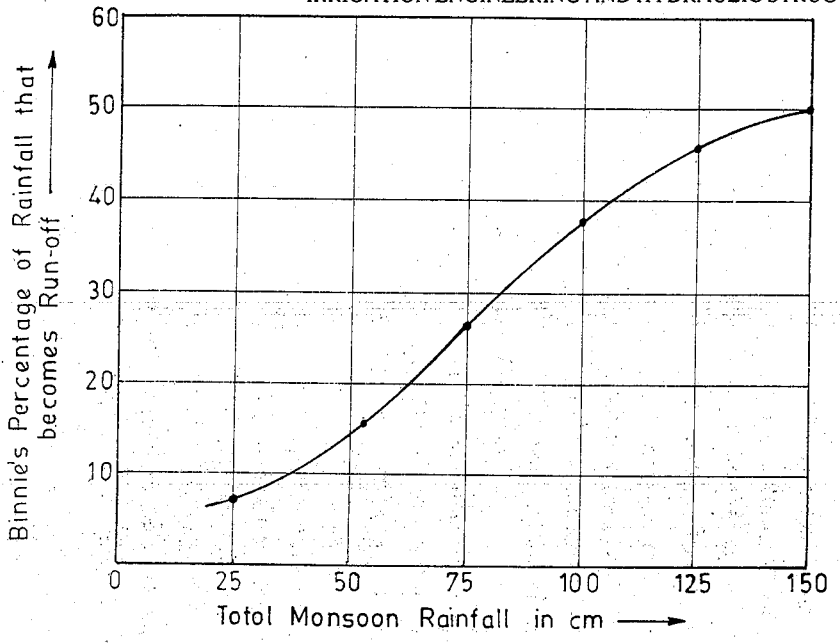


Fig. 18.6(*a*). Binnie's Monsoon Rainfall-runoff Curve.

(2) **Strange's percentages and tables.** Mr. W.L. Strange carried out investigations on catchments in Bombay Presidency, and worked out percentages for converting monsoon rainfall into monsoon yield. He even worked out such percentages for converting daily rainfalls into daily runoffs. It was an improvement over Binnie's tables, since he divided the catchments into three categories to account for the general characteristics of the catchments. The catchments prone to producing higher yields, such as those with more paved areas, etc. were categorised as *good catchments;* and those prone to producing low yields were termed as *bad catchments.* The intermediate types were called *average catchments.* Different runoff percentages were given for different types of catchments for different values of rainfalls, as shown in table 18.6(*a*). These values have also been drawn in the shape of curves, as shown in Fig. 18.6(*b*).

Table 18.6. Values of Strange's Run off Percentages

Monsoon Rainfall in cm	*Runoff Percentages for catchments designated as*			*Monsoon Rainfall in cm*	*Runoff Percentages for catchments designated as*		
	Good	*Average*	*Bad*		*Good*	*Average*	*Bad*
25.0	4.3	3.2	2.1	107.5	40.9	30.6	20.4
30.0	6.2	4.6	3.1	110.0	42.0	31.5	21.0
35.0	8.3	6.2	4.1	112.5	43.1	32.3	21.5
40.0	10.5	7.8	5.2	115.0	44.3	33.2	22.1
45.0	12.8	9.6	6.4	117.5	45.4	34.0	23.2
50.0	15.0	11.3	7.5	120.0	46.5	34.8	23.8
55.0	17.3	12.9	8.6	122.5	47.6	35.7	23.8
60.0	18.5	14.6	9.7	125.0	48.8	36.6	24.4
65.0	21.8	16.3	10.9	127.5	49.9	37.4	24.9
70.0	24.0	18.0	12.0	130.0	51.0	38.2	25.5
75.0	26.3	19.7	13.1	132.5	52.0	39.0	26.0
80.0	28.5	21.3	14.2	135.0	53.3	39.9	26.6
85.0	30.8	23.1	15.4	137.5	54.4	40.8	27.2
90.0	33.0	24.7	16.5	140.0	55.5	41.6	27.7
95.0	35.3	26.4	17.6	142.5	56.6	42.4	28.3
100.0	37.5	28.1	18.7	145.0	57.8	43.3	28.9
102.5	38.5	28.9	19.3	147.5	58.9	44.1	29.4
105.0	39.5	29.8	19.9	150.0	60.0	45.0	30.0

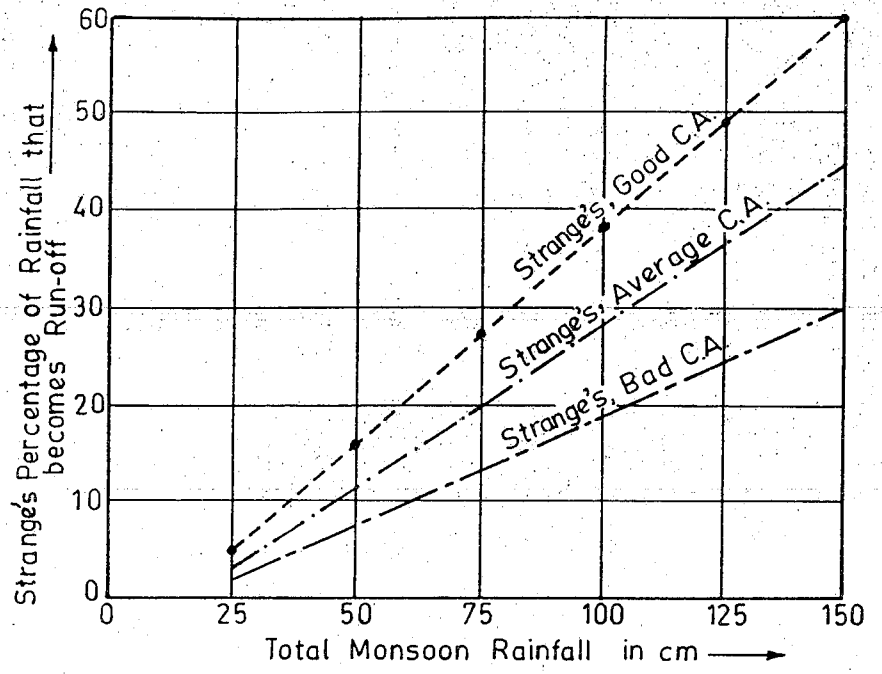


Fig. 18.6(*b*). Strange's monsoon Rainfall-Runoff curves.

(3) **Barlow's tables.** Mr. T.G. Barlow, the first Chief engineer of Hydro-Electric Survey of India, carried out extensive investigations on catchments mostly under 130 square km in U.P. State of India. On the basis of his investigations, he divided catchments into five classes and assigned different percentages of rainfall that become runoff (over long periods) for each class of catchment, as indicated in table 18.7.

Table 18.7. Values of Barlow's Runoff Percentages

S.No.	*Class of catchment*	*Description of the catchment*	*Runoff percentages (K)*
1.	A	Flat, cultivated, adsorbent soils	10
2.	B	Flat, partly cultivated stiff soils	15
3.	C	Average catchment	20
4.	D	Hills and plains with little cultivation	25
5.	E	Very hilly and steep catchment with little or no cultivation	33

The above values of runoff percentages are for the average type of monsoons, and are to be modified by the application of the following coefficients (Table 18.8) according to the nature of the season.

Table 18.8. Values of Barlow's coefficient to be multiplied with K values of table 18.7 to obtain True Runoff Percentages

S.No.	*Nature of season*	*Class of catchment*				
		A	*B*	*C*	*D*	*E*
1.	Light rain, no heavy down pour	0.7	0.8	0.8	0.8	0.8
2.	Average or varying rainfall, no continuous down pour	1.0	1.0	1.0	1.0	1.0
3.	Continuous down pour	1.5	1.5	1.6	1.7	1.8

(4) **Lacey's formula.** This formula connects the monsoon *rainfall (P) with the yield (Q)* by the equation.

$$\text{Yield } (Q) = \left[\frac{P}{1 + \frac{304.8\,m}{p.n}}\right] \qquad ...(18.5)$$

where m = a constant, called *monsoon duration factor,* the values of which are given in table 18.9.

n = a constant, called *catchment factor,* the values of which are given for different classes of catchments (as defined by Barlow) in table 18.10.

Table 18.9. Values of m to be used in Eq. (18.5)

S.No.	*Duration of Monsoon*	*Monsoon Duration factor m*
1.	Bad year	0.5
2.	Normal year	1.2
3.	Good year	1.5

Table 18.10. Values of *n* to be used in Eq. (18.5)

S.No.	*Class of catchment*	*Description of the catchment*	*Values of n*
1.	A	Flat cultivated, absorbent soils	0.25
2.	B	Flat, partly cultivated stiff soils	0.60
3.	C	Average catchments	1.00
4.	D	Hills and plains with little cultivation	1.70
5.	E	Very hilly and steep catchments with little or no cultivation	3.45

(5) **Inglis formula.** Inglis derived his formula for catchments of West Maharashtra State of India. He divided the areas as ghat areas (*Sahyadri ranges*) where rainfall is 200 cm or more; and non-ghat areas where rainfall is less than 200 cm. His formulas are:

(*a*) *For ghat areas, with rainfall (P) equalling or exceeding 200 cm:*

$$\text{Yield} = (0.85P - 30.48) \text{ cm} \qquad ...(18.6)$$

where P is the rainfall in cm.

(*b*) *For non-ghat areas with rainfall P less than 200 cm.*

$$\text{Yield} = \frac{P\,(P - 17.78)}{254} \text{ cm} \qquad ...(18.7)$$

where P is the rainfall in cm.

(6) **Khosla's formula.** This formula is based upon the recent research work conducted in this field, and is a very simple and useful formula. It can be easily applied to the entire country, without bothering for the region of its origin. This formula states that

$$\text{Yield } (Q) = P - 0.48\,T_m \qquad ...(18.8)$$

where Q = the yield in cm

P = the rainfall in cm

T_m = mean annual temperature of the area.

Example 18.4. *The design annual rainfall for the catchment of a proposed reservoir has been computed to be 99 cm. The catchment area has been estimated to have the mean annual temperature of 20°C. The catchment area contributing to the proposed reservoir is 1000 sq.km. Calculate the annual design catchment yield for this reservoir. Make use of Khosla's formula.*

Solution. Use Khosla's formula connecting the design rainfall (P) with the design yield (Q) by Eq. (18.8) as:

$$Q = P - 0.48\, T_m$$

where $P = 99$ cm (given)

$T_m = 20°C$ (given)

$$\therefore \quad Q = 99 - 0.48 \times 20 = 89.4 \text{ cm} = \mathbf{0.894\ m}$$

The total yield produced in m^3 from the given catchment of 1000 sq.km (*i.e.* $1000 \times 10^6\ m^2$)

$$= 0.894 \times 1000 \times 10^6\ m^3 = 894\ M.m^3 \quad \textbf{Ans.}$$

18.4.2. Use of Flow Duration Curves for Computing Dependable Flow

As we know, the stream flow varies widely over a **water year***. This variability of stream flow can be studied by plotting *flow duration curves* for the given stream. A **flow duration curve**, also called as **discharge-frequency curve**, is a curve plotted between *stream flow (Q)* and *percent of time the flow is equalled or exceeded (P_p)*, and is of the type shown in Fig. 18.7 (*a*), (*b*) and (*c*).

Such a curve can be plotted by first arranging the stream flow values (Q) in descending order using class intervals, if the number of individual values is very large. The data to be used can be of daily values, weekly values, or monthly values. If N data values are used, the plotting position of any discharge (or class value) Q is given, as :

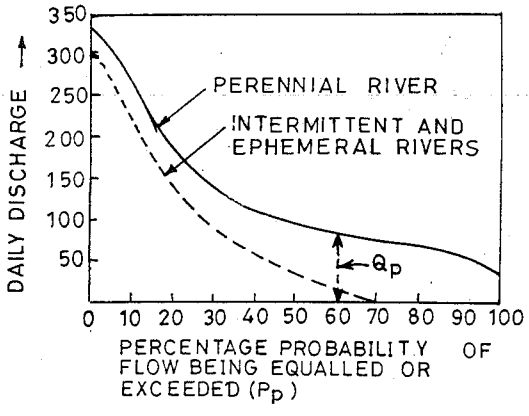


Fig. 18.7(*a*). Typical flow-duration curves on an ordinary paper.

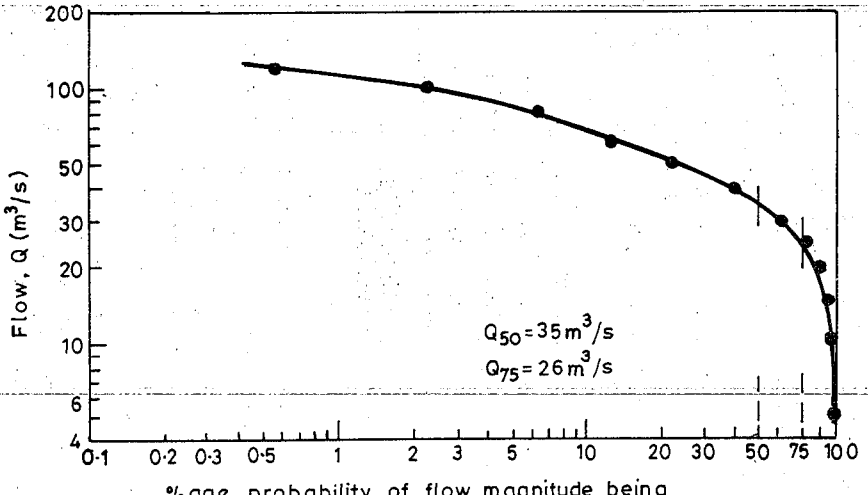


Fig. 18.7(*b*). A typical flow duration curve on a log-log paper.

* 1st June to 30st May.

$$P_p = \frac{m}{N+1} \times 100\%$$

where m = is the order No. of that discharge (or class value)

P_p = percentage probability of the flow magnitude being equalled or exceeded.

The ordinate Q at any percentage probability p (such as 60%), *i.e.* Q_p, will represent the flow magnitude of the river that will be available for 60% of the year, and is hence termed as *60% dependable flow* (Q_{60}). Q_{100} (*i.e.* 100% dependable flow) for a perennial river can, thus, be read out easily from such a curve. Q_{100} for an *ephmeral* or for an *intermittent river* shall evidently be zero.

A flow-duration curve represents the cumulative frequency distribution, and can be considered to represent the stream flow variation of an average year. Such a curve can be plotted on an ordinary arithmetic scale paper, or an semi-log or log-log paper. The following *characteristics of flow-duration curves* have been noticed.

(1) The slope of the flow-duration curve depends upon the interval of data used. Say for example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same river. This happens due to smoothening of small peaks in monthly data.

(2) The presence of a reservoir on a stream upstream of the gauging point will modify the flow-duration curve for the stream, depending upon the reservoir-regulation effect on the released discharges. A typical reservoir regulation effect on flow duration curve is shown in Fig. 18.7 (*c*).

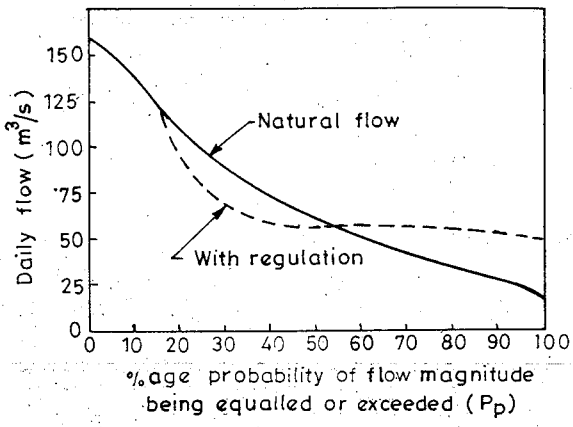


Fig. 18.7. (*c*) Reservoir regulation effect on the flow-duration curve of a stream.

(3) The flow-duration curve, when plotted on a log probability paper, is found to be a straight line at least over the central region, as shown in Fig. 18.7 (*c*). From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.

(4) The flow-duration curve plotted on a log-log paper (Fig. 18.7 (*b*)) is useful in comparing the flow characteristics of different streams. Say for example, a steep slope of the curve indicates a stream with a highly variable discharge ; while a flat slope of the curve indicates a small variability of flow and also a slow response of the catchment to the rainfall. A flat portion on the lower end of the curve indicates considerable base flow. A flat portion on the upper end of the curve is typical of river basins having large flood plains, and also of rivers having large snowfall during a wet season.

(5) The chronological sequence of occurrence of the flow gets hidden in a flow duration curve. A discharge of say 500 m^3/s in a stream will, thus, have the same percentage probability (P_p), irrespective of whether it occurred in January or June. This aspect, a *serious handicap of such curves*, must be kept in mind while interpreting a flow-duration curve.

Flow-duration curves find a considerable use in water-resources planning and development activities. Some of their important uses are indicated below :

(*i*) For evaluating dependable flows of various percentages, such as 100%, 75%, 60%, etc. in the planning of water-resources engineering projects.

(*ii*) In evaluating the characteristics of the hydropower potential of a river.

(*iii*) In comparing the adjacent catchments with a view to extend the stream flow data.

(*iv*) In computing sediment load and dissolved solids load of a stream.

(*v*) In the design of drainage systems, and

(*vi*) in flood control studies.

18.5. Fixing the Reservoir Capacity for the Computed Value of the Dependable Yield of the Reservoir Catchment

After deciding the dependable yield for the proposed reservoir (tank), the reservoir capacity is decided as follows:

The water demand (annual of course) is computed by estimating the *crop water requirement* (including transit losses) and any other water demand required to meet the water supply needs, or the downstream commitments of water release, if any. Reservoir losses @ about 15% of the water demand is then added to obtain the *live* or the *net storage* required to meet the given demand. *Dead storage* is now added to this live storage to obtain the *gross storage* required to meet the demand. The *reservoir capacity*, however, cannot exceed the *catchment yield* (inflow into the reservoir), and hence the reservoir capacity is fixed at a value which is lesser of the value of the assessed gross storage required to meet the demand; and ii) the assessed dependable yield for the reservoir site. The full tank level (FTL) or the full reservoir level (FRL) is finally computed from the elevation-capacity curve.

The *dead storage* level or dead storage required in the above computation is usually fixed at higher of the following values:

(*a*) dead storage = rate of silting x Life of the reservoir

(*b*) dead storage = 10% of gross storage or net water demand.

(*c*) dead storage level being equal to the full supply level of the off taking canal at the tank site

Example 18.5. *The lowest portion of the capacity-elevation curve of a proposed irrigation reservoir, draining 20 km^2 of catchment, is represented by the following data:*

Elevation in m	*Capacity in ha.m*
RL 600	*24.2*
602	*26.2*
604	*30.3*
606	*36.8*

The rate of silting for the catchment has been assessed to be 300 m^3/km^2/year. Assuming the life of the reservoir to be 50 years, (a) compute the dead storage, and the lowest sill level (LSL), if the main canal is 6km long with a bed slope of 1 in 1000, and the canal bed level at the tail end is at RL 594.5 m. The FSD of the canal at the head is 80 cm. The crop water requirement is assessed as 250 ham.

(b) If the dependable yield of the catchment is estimated to be 0.29m, what will be the gross capacity of the reservoir ?

Solution. The *dead storage* is first of all computed as maximum of the following three values :

(*a*) dead storage = rate of silting × life of the reservoir

$$= 300\ \text{m}^3/\text{km}^2/\text{year} \times \underset{\text{(C.A.)}}{20\ \text{km}^2} \times \underset{\text{(Life)}}{50\ \text{year}} = 300000\ \text{m}^3$$

$$= 30\ \text{ha.m.} \qquad ...(i)$$

(*b*) dead storage

= 10% × net water demand or crop water requirement = 10% × 250 ha.m

= 25 ha.m ...(*ii*)

(*c*) Dead Storage Level = FSL of canal at headworks

$$= 594.5 + (6 \times 1000)\frac{1}{1000} + 0.8 = 601.4 \text{ m}$$

Dead Storage Capacity at RL 601.4 m is interpolated

$$= 24.2 + \frac{(26.2 - 24.2) \text{ ham}}{RL\ (602 - 600)} \times RL\ (601.4 - 600) = 24.2 + \frac{2 \times 1.4}{2}$$

$$= 24.2 + 1.4 \text{ ham} = 25.6 \text{ ham} \qquad ...(iii)$$

The dead storage is fixed at maximum of the three values obtained at (*i*), (*ii*) and (*iii*) above, *i.e.* 30 ham, 25 ham, and 25.6 ham. Hence, choose the dead storage at **30 ham. Ans.**

The *LSL* (x) corresponds to 30 ham capacity, which is computed as :

$$30 = 26.2 \text{ ham} + \frac{(30.3 - 26.2) \text{ ham}}{RL\ (604 - 602)} \times RL\ (x - 602)$$

or $$3.8 = \frac{4.1}{2}(x - 602)$$

$$(x - 602) = \frac{3.8 \times 2}{4.1} = 1.85$$

$$x = RL\ \mathbf{603.85\ m.}$$

The lowest sill level *i.e.* dead storage level is thus fixed at **RL 603.85 m. Ans.**

(*b*) Net water demand = Crop water requirement including transit losses = 250 ham

Reservoir losses = 15% × 250 ham = 37.5 ham

Live storage required

to meet the given demand = (250 + 37.5) ham = **287.5 ham**

Dead storage = 30 ham (computed above)

∴ Gross reservoir storage required

= Live storage + Dead storage = 287.5 + 30 = **317.5 ham**

Dependable yield = 0.29 m (depth) = 0.29 m × (20×10^6 m^2) = **580 ham**

The gross capacity of the reservoir is fixed at the lesser of the gross storage required to meet the demand (*i.e.* 317.5 ham) and the dependable yield (*i.e.* 580 ha.m.). Hence, the reservoir capacity = **317.5 ham. Ans.**

18.6. Relation between Inflow, Outflow, and storage Data for a Reservoir

The *inflow* to the reservoir and the *outflow* from the reservoir are the only two factors which govern the storage capacity of a reservoir. Since the inflow to the reservoir is variable, water is stored in the reservoir to cater to the required outflow from the reservoir, particularly during the critical periods in non-monsoon season. Naturally, if more outflow is required, more capacity has to be provided.

As a matter of fact, after assessing the monthly or annual inflows into the reservoir and representing it by the mass inflow curve, the demand pattern is specified. The reservoir is then usually designed to meet this specified demand, represented by the mass outflow curve.

The reservoir capacity, the reservoir inflow, and the outflow from the reservoir are governed by the storage equation, given by:

Inflow – Outflow = Increase in storage

∴ Increase in Reservoir Storage = Inflow – Outflow ...(18.9)

18.7. Fixing the Reservoir Capacity from the Annual Inflow and Outflow Data

The capacity of reservoir may be determined by determining the storage needed to accommodate the given inflow minus the given outflow, as governed by the above equation. However, this study involves numerous factors, as discussed below:

Streamflow data at the reservoir site must be known. Monthly inflow rates are sufficient for large reservoirs, but daily data may be required for small reservoirs. When the inflow data at the dam site are not known, the data at a station elsewhere

on the stream or on a nearby stream may be collected and adjusted to the dam site. The available data may sometimes be extended so as to include a really drought period.

Besides determining the streamflow data at the dam site, an adjustment has to be made for the water required to be passed from the reservoir to satisfy the prior water rights and to obey the agreements between various sharing States through which the river is passing.

Moreover, the construction of a reservoir increases the exposed area of the water surface above that of the natural stream, and thus, increases the evaporation losses. There is, thus, a net loss of water occurring due to reservoir construction, Sometimes, these losses may be so huge that the entire purpose of the reservoir may be defeated. Seepage from the reservoir may also add to the loss resulting from the reservoir.

All these factors make this study very very complex. An approximate easy solution for determining the reservoir capacity may be obtained graphically with the help of *mass curves* as explained in the next article. On the other hand, a tabular solution is necessary in order to account for all important factors. For more precise results and for all complex systems, computers may be used for programming the analysis (called operation study).

Example 18.6. *Monthly inflow rates during a low-water period at the site of a proposed dam are tabulated in Col. (2) of table 18.9. The corresponding monthly pan evaporation and precipitation at a nearby station are also tabulated in Col. (3) and Col. (4) of the same table. Prior water rights make it obligatory to release the full natural flow or 15 hectare-metres per month, whichever is minimum. If the estimated monthly demands are as given in col. (5) of table 18.9 and the net increased pool area is 400 hectares, find the required storage capacity for the reservoir. Assume pan evaporation coefficient=0.7 and also assume that only 28% of the rainfall on the land area to be flooded by reservoir has reached the stream in the past.*

Table 18.11

Month	*Inflow at dam site in hectare metres*	*Pan evaporation in cm*	*Precipitation in cm*	*Demand in hectare-metres*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
January	1.2	1.8	1.3	15.8
February	0.0	1.8	1.7	14.3
March	0.0	2.6	0.6	9.6
April	0.0	10.2	0.0	4.8
May	0.0	15.4	0.0	3.5
June	0.0	1.6	1.1	3.4
July	240.0	10.8	16.1	5.0
August	480.0	11.7	16.4	5.0
September	1.0	10.8	2.2	10.0
October	0.6	9.6	0.8	15.6
November	0.5	7.8	0.0	16.8
December	0.2	2.0	0.0	16.8
Σ	723.5	101.1	40.2	120.6

Solution. Table 18.11 is extended as shown in Table 18.12. The 28% precipitation is already reaching and is included in the given inflow [Col. (2)], and hence, only 72% of the precipitation is to be included, as worked out in Col. (8). The table is otherwise self- explanatory and the monthly water drawn from the reservoir is worked out in Col. (10).

Table 18.12. Solution Table for Example 18.6

Month	*Inflow at dam site in hectare-metres*	*Pan evapo-ration in cm*	*Precipi-tation in cm*	*Demand in hectare-metres*	*Require-ment due to prior rights equal to Col. (2) or 15 ha.m. whichever is minimum*	*Evaporation in hectare-metres* $\frac{400 \times Col.\ (3)}{100} \times 0.7$ $= 2.8 \times Col.(3)$	*Precipitation in hectare metres* $\frac{400 \times Col.\ (4)}{100} \times 0.72$ $= 2.88 \times Col.(4)$	*Adjusted inflow. Col. (2) + Col. (8) – Col. (6) – Col. (7) in hectare metres*	*Water required from storage in ha. m. Col. (5) – Col. (9) (only +ve values)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
January	1.2	1.8	1.3	15.8	1.2	5.03	3.74	(–) 1.29	17.09
February	0.0	1.8	1.7	14.3	0.0	5.03	4.9	(–) 0.13	14.43
March	0.0	2.6	0.6	9.6	0.0	7.28	1.73	(–) 5.55	15.15
April	0.0	10.2	0.0	4.8	0.0	28.56	0.0	(–) 28.56	33.36
May	0.0	15.4	0.0	3.5	0.0	43.1	0.0	(–) 43.1	46.60
June	0.0	16.6	1.1	3.4	0.0	46.4	3.17	(–) 43.23	46.63
July	240.0	10.8	16.1	5.0	15.0	30.3	46.4	(+) 241.1	Nil
August	480.0	11.7	16.4	5.0	15.0	32.8	47.3	(+) 479.5	Nil
September	1.0	10.8	2.2	10.0	1.0	30.3	6.34	(–) 23.96	33.96
October	0.6	9.6	0.8	15.6	0.6	26.9	2.31	(–) 24.39	39.99
November	0.5	7.8	0.0	16.8	0.5	21.8	0.0	(–) 21.8	38.60
December	0.2	2.0	0.0	16.8	0.2	5.6	0.0	(–) 5.6	22.40
Σ	723.5	101.1	40.2	120.6	33.5	283.1	116.49		308.21

Finally, their summation (*i.e.* **308.21 hectare-metres**) works out to be the required storage capacity of the reservoir. **Ans.**

18.8. Fixation of Reservoir Capacity with the Help of Mass Curves of Inflow and Outflow

After the flow hydrographs for the stream at the dam site have been plotted for a large number of years (say 25 to 30 years), the required storage capacity for a reservoir with a given outflow pattern can be approximately calculated with the help of mass curves. A hydrograph is a plot of discharge vs. time, while a *mass curve* is a plot of accumulated flow vs. time. The area under the hydrograph between times $t = 0$ and $t = t$ will represent nothing but the accumulated flow up to the time t, and hence, the ordinate of the mass curve at time *t*.

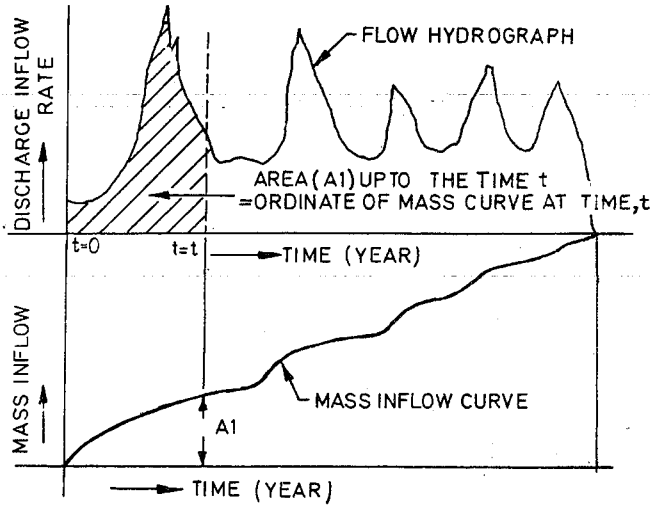


Fig. 18.8 (*a*) and (*b*).

A typical annual inflow hydrograph is shown in Fig. 18.8 (*a*), and the mass curve for this inflow hydrograph is plotted in Fig. 18.8 (*b*). The area under the first curve upto a time (*t*) is equal to the ordinate of the second curve at the same time (*t*). Adjustments for the scales and units of the two curves must be made while plotting.

It is evident that a mass curve will continuously rise, as it is the plot of the accumulated inflow. Periods of no inflow would be represented by the horizontal lines on the mass inflow curve. To differentiate such a mass curve of runoff from the mass curve of rainfall, this mass curve is usually called as the **flow mass curve** and is an integral of the flow hydrogrph.

The mass curve may also be called the *ripple diagram.* The slope of the mass curve at any time is a measure of the inflow rate at that time.

After the inflow mass curve has been plotted, the mass curve of demand may also be plotted by accumulating the required outflow. *If a constant rate of withdrawal is required from the reservoir, the mass curve of demand will be a straight line having a slope equal to the demand rate.* Demand curves or demand lines are generally straight lines (representing uniform withdrawal) although, in practice, they may be curved also [See Fig. 18.9 (*a*) and (*b*)].

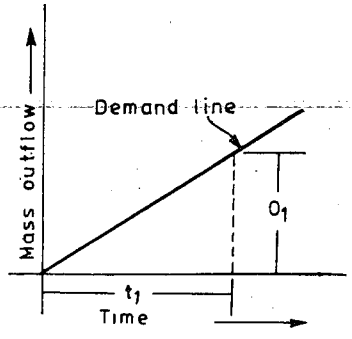


Fig. 18.9 (*a*)

O_1/t_1 = Slope of the line = Demand rate

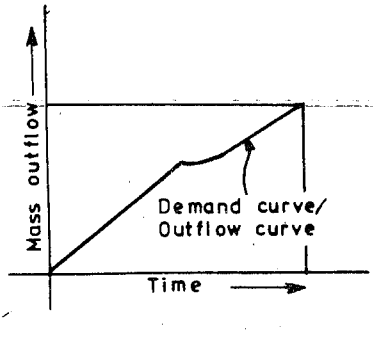


Fig. 18.9 (*b*).

Determining Reservoir Capacity for a Given Demand

The mass curve of inflow and the demand line can be used to determine the required storage capacity. In Fig. 18.10, it is evident that the demand lines drawn tangent to the high points A_1, A_2, A_3... of the mass curve, represent the rate of withdrawal from the reservoir. Assuming the reservoir to be full whenever the demand line intersects the mass curve (points F_1, F_2 ...), the maximum departure (B_1C_1, B_2C_2...) between the two curves represents the reservoir capacity just required to satisfy the demand.

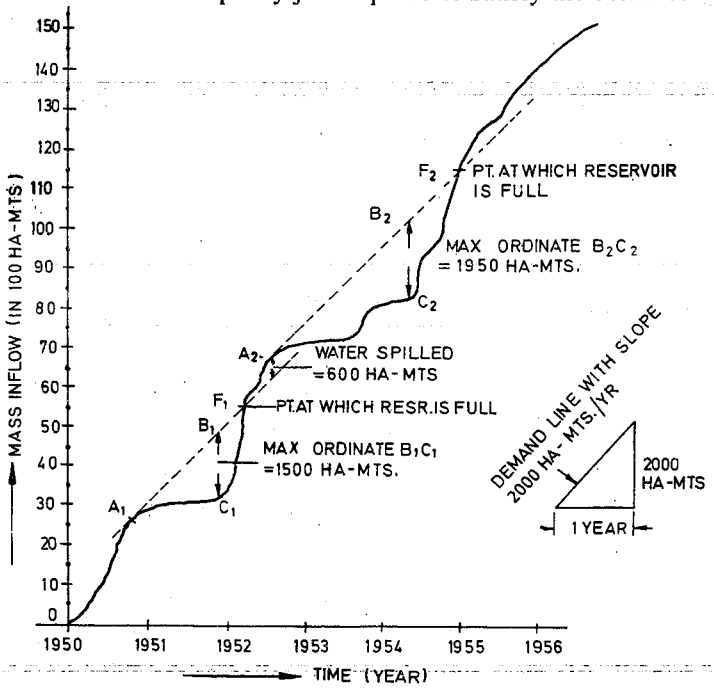


Fig. 18.10.

Fig. 18.10 has been drawn for a demand of 2,000 hectare- metres/year. The biggest departure ordinate (*i.e.* the maximum of B_1C_1, B_2C_2...) works out to be 1,950 hectare-metres/year, which represents the required storage capacity for the reservoir.

The vertical distance between the successive tangents A_1B_1, and A_2B_2, etc., represent the water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

For Fig. 18.10, the spillway capacity works out to be 600 hectare-metres, and the reservoir capacity as 1,950 hectare-metres. It can also be observed that:

(*i*) Assuming the reservoir to be full at A_1, it is depleted to 1,950 – 1,500 = 450 ha-m at C_1 and is again full at F_1.

(*ii*) The reservoir is full between F_1 and A_2, and the quantity of water spilled over the spillway is equal to 600 ha-m.

(*iii*) From A_2, the water starts reducing in the reservoir till it becomes fully empty at C_2.

(*iv*) The water again starts collecting in the reservoir and it is again full at F_2.

Note 1. It may also be noted that a demand line, when extended, must intersect the mass curve. If it does not, the reservoir will not refill.

Note 2. When the demand curve is not a straight line, then the two mass curves are superimposed over each other in such a way that their origins and axis coincide. The larger ordinate between the two, gives the required storage capacity, as shown in Fig. 18.11.

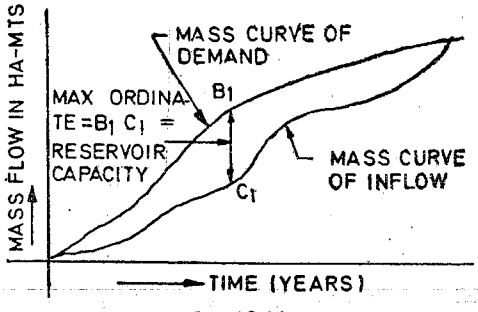


Fig. 18.11.

Fixing the Demand for a Reservoir of a Given Capacity

In the previous article, we have explained as to how the reservoir capacity can be determined for a given demand. The reverse, *i.e.* fixing the demand for a given reservoir capacity, may also be done with the help of mass curve of inflow.

In this case, the tangents are drawn to the high points (A_1, A_2, ...) of the mass inflow curve in such a way that the maximum departure from the mass curve is equal to the reservoir capacity. The slopes of the lines so drawn represent the demand rate which can be obtained with this capacity during different periods. The minimum value of these slopes will represent the withdrawal rate, which can certainly be obtained from the given reservoir, and will, thus, represent its *firm yield.*

For example in Fig. 18.12, *a mass inflow curve is given. It is further required to find out the possible yields and the safe yield for a reservoir capacity of 750 ha-m.* To determine these values, we shall proceed as follows :

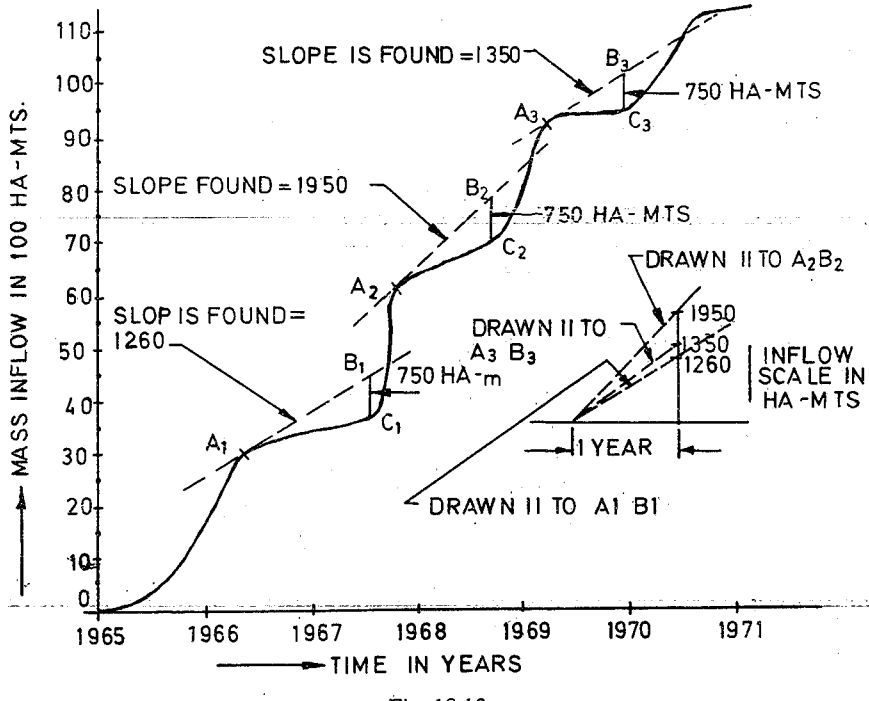


Fig. 18.12.

First of all, the high points A_1, A_2 and A_3 are marked on the mass inflow curve. The points B_1, B_2, and B_3 are determined in such a way that their maximum departure from the curve is equal to 750 ha-m. The tangent lines A_1B_1, A_2B_2 and A_3B_3 are then drawn. The slopes of these lines are determined and are found to be 1,260, 1,950, 1,350

ha-m/year, respectively. These values represent the possible yields in different periods. *The minimum of them, i.e.* 1,260 ha-m/year represents the safe yield or the firm yield of the reservoir. **Ans.**

Example 18.7. *Annual runoff in terms of depth over the catchment area of 1675 sq.km. of a reservoir is given below :*

Year	*1962*	*63*	*64*	*65*	*66*	*67*	*68*	*69*
Runoff (cm)	*98*	*143.5*	*168.3*	*94*	*95.3*	*152.4*	*110*	*131.3*

Draw the flow mass diagram. What is the average yield from the catchment? What should be the live storage capacity of the reservoir to use the source fully? If the dead storage is 20% of the live storage, what is the gross storage? Mark the filling and emptying periods on the mass curve. (*Bhopal University, 1980*)

Solution. The cumulative runoff values are worked out in Col. 4 of table 18.13, and they are plotted against the values of corresponding years (col. 1) of the same table, so as to obtain the desired mass diagram [Fig. 18.13].

Table 18.13

Year	*Yearly runoff (cm)*	*Runoff as volume in* $M.m^3 = \frac{Col.\ (2)}{100} \times 1675$	*Cumulative runoff as volume in* $M.m^3$
(1)	*(2)*	*(3)*	*(4)*
1962	98	1642	1642
63	143.5	2404	4046
64	168.3	2819	6865
65	94	1575	8440
66	95.3	1596	10,036
67	152.4	2553	12,589
68	110	1842	14,431
69	131.3	2199	16,630
	$\Sigma = 992.8$	16630	

The **average annual yield** of the catchment is the arithmetic mean of the given annual yields, and is equal to

$$= \frac{992.8}{8} = 124.1 \text{ cm of runoff}$$

$$= 1.241 \text{ m} \times CA \text{ in m}^2 \qquad \text{(volume of runoff)}$$

$$= 1.241 \text{ m} \times (1675 \times 10^6) \text{ m}^2$$

$$= 2078.68 \text{ Mm}^3; \text{ say } \mathbf{2079\ Mm^3} \quad \textbf{Ans.}$$

Now, to utilise the source fully, there should not be any spilling over of the water, and the yearly demand should be equal to the average yield *i.e.* 2079 Mm^3.

To determine the required reservoir capacity to meet this demand rate, a line is drawn from the high point A_1, parallel to this demand rate, as shown in Fig. 18.13; and the maximum departure of this line from the mass curve is read out as $B_1C_1 = 1065\ Mm^3$.

Hence, the required reservoir capacity is 1065 Mm^3. **Ans.**

Physically speaking, the reservoir will be empty at trough pts C_1 and C_2, and full at ridge point A_1.

Alternatively, the mass demand line can be plotted, and both curves extended back up to the origin O, as shown in Fig. 18.14.

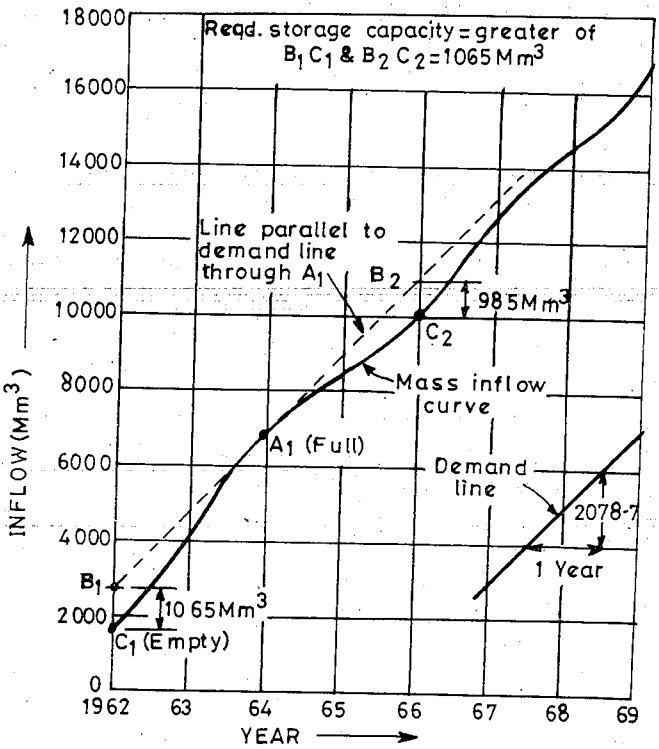


Fig. 18.13.

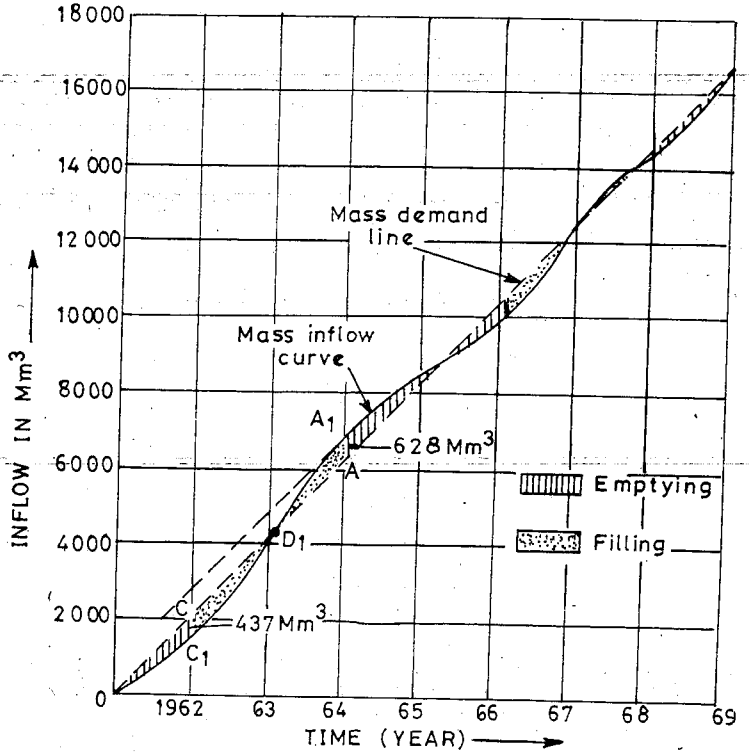


Fig. 18.14.

It can be seen from this curve, that from O to C_1, the slope of the inflow curve is less than that of the demand curve, indicating that the inflow is less than the outflow, and the reservoir is *emptying*. From C_1 to D_1, the slope of the inflow curve is in excess of the demand, and reservoir is *filling*. The vertical intercept $C_1 C$, from the inflow curve at C_1 to the demand curve at C, represents the initial storage to meet the demand from O to C_1. At D_1, the reservoir level is the same as at O. After D_1, and upto A_1, the slope of the inflow curve is steeper than that of the demand curve, and as such, the reservoir is still rising. Point A_1 represents the full reservoir level. Vertical intercept A_1A represents the storage between the initial water level corresponding to the point O and the full reservoir level. The total minimum storage capacity of the reservoir to meet the demand is thus given by the vertical intercepts C_1C and A_1A.

Max. withdrawl from storage + Max. stored in storage

$= 437 + 628 = 1065$ mcm. **Ans.**

The emptying and filling processes are also shown in Fig. 18.14 ; filling will occur when the slope of the inflow curve is more than that of the demand curve, and *vice versa.*

18.9. Fixation of Reservoir Capacity Analytically using Sequent Peak Algorithm

The *sequent peak algorithm* is a simple and straight forward analytical procedure, for computing reservoir capacity, and is used as an excellent alternative to the mass curve method of determining reservoir capacity.

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period, and storage required to pass the dry period is estimated. If the mass curve contains only one ridge point, and if there is no well defined subsequent trough point, it may become necessary to repeat the given data for one more cycle to arrive at the desired storage determination. Also, the demand rate is usually not a straight line (as assumed in mass curve analysis), since the demand (out flow) generally becomes non-uniform due to seasonal variations in the demand. *The sequent peak algorithm technique helps us to device simple mathematical solution to the problem of computing the reservoir capacity.*

Sequent peak algorithm (Fig. 18.15) is a plot between time (say, in months) on X-axis, and cumulative inflow minus cumulatives outflow on Y-axis. The quantity taken on Y-axis, corresponding to each month equals to [Σ Inflow $-$ Σ outflow] = [Σ (Inflow-Outflow]. This value is also called *cumulative net inflow.*

The $+$ *ve* values of cumulative net inflow, representing cumulative surplus of inflow will be plotted above X-axis, while its negative values, representing cumulative deficit of inflow, will be plotted below x-axis.

The obtained plot will consist of peaks and troughs, as shown in Fig. 18.15.

The first ridge point A_1 in this plotting (*i.e.* first peak of cumulative net surplus) is called the *first peak* ; while subsequent ridge points A_2, A_3, A_4, etc., are called *sequent peaks.*

Similarly, the first trough point B_1 is called the first trough, while subsequent trough points B_2, B_3, etc. may be called the *sequent troughs.*

The difference between the first peak and first trough (height $A_1 B_1$) will, in this plot, evidently, represent the reservoir storage required under normal inflows, and

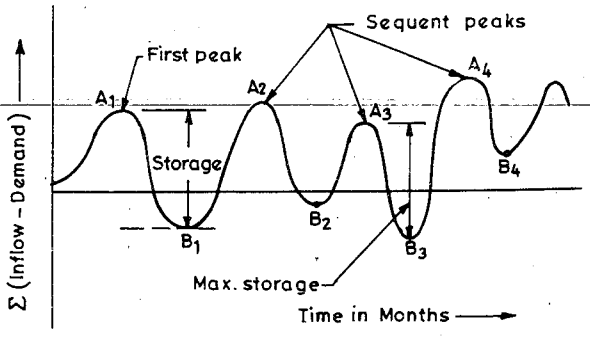


Fig. 18.15. Sequent Peak Algorithm.

is called the *normal storage* ; whereas, the maximum difference between any sequent peak and the just following trough will represent the *maximum storage* required for the reservoir.

The normal and maximum storage through sequent peak algorithm is calculated as follows :

(1) Convert the monthly inflows into the volume units for the period of the available data.
(2) Estimate the monthly volumes of all the outflows from the reservoir. This should include losses from evaporation, seepage, and other losses.
(3) Compute the cumulative values of Inflows.
(4) Compute the cumulative values of outflows.
(5) Compute the values of cumulative inflow minus cumulative outflow ; *i.e.* [Σ Inflow – Σ Outflow].
(6) Plot a graph by taking months (time) on X-axis, and $\Sigma (I - 0)$ of step (5) on Y-axis, on an ordinary graph paper.
(7) The data will plot peaks and troughs. The second and subsequent peaks are called sequent peaks.
(8) The maximum difference between any sequent peak and the just following trough is the maximum storage required for the reservoir. The difference between the first peak and the trough following it, is the storage required under normal inflows.

The method will become more clear on solving example 18.8.

Example 18.8. *Monthly inflows at a proposed reservoir site for a drought period of 15 months are given along with targetted demands (found from a working table) in the table below. Compute the storage required by plotting sequent peak algorithm.*

Table 18.14

Month	*June*	*July*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*	*Jan*	*Feb*	*March*	*April*	*May*	*June*	*July*	*Aug*
River Inflows ($M\,m^3$)	250	350	400	200	150	150	100	50	150	300	400	450	150	200	450
Targetted demand $M\,m^3$	150	150	200	250	350	400	250	200	150	150	100	250	350	300	100

Solution. Calculations are carried out in Table 18.15 to compute $\Sigma (I - O)$ in col (6).

Table 18.15

Month	*Inflow (I) $M\,m^3$*	*Outflow (O) $M\,m^3$*	*Cumulative Inflow $\Sigma I\,(M\,m^3)$*	*Cumulative Outflow $\Sigma O\,(M\,m^3)$*	$(\Sigma I - \Sigma O)$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
June	250	150	250	150	+ 100
July	350	150	600	300	+ 300
Aug	400	200	1000	500	+ 500
Sept	200	250	1200	750	+ 450
Oct	150	350	1350	1100	+ 250
Nov	150	400	1500	1500	0
Dec	100	250	1600	1750	(–) 150
Jan	50	200	1650	1950	(–) 300
Feb	150	150	1800	2100	(–) 300
March	300	150	2100	2250	(–) 150
April	400	100	2500	2350	+ 150
May	450	250	2950	2600	+ 350
June	150	350	3100	2950	+ 150
july	200	300	3300	3250	+ 50
Aug	450	100	3750	3350	+ 400

To calculate values of $\Sigma (I - O)$, values of ΣI (cumulative inflow) are written in col (4), and values of ΣO (cumulative outflow) are written in col (5).

Values of $(\Sigma I - \Sigma O) = \Sigma (I - O)$ are new calculated by subtracting values of col (5) from values of col (4). These values are written in col (6) with their $+ve$ or $-ve$ sign, representing excess & cumulative deficit, respectively. When values of $(\Sigma I - \Sigma O)$ from col (6) are plotted on y-axis along with corresponding values of month (col 1) taken on x-axis, a plot containing peaks and troughs will be obtained, as shown in Fig. 18.16, called sequent peak algorithm. The difference between first peak and the trough following it, is found to be **800 M m^3**, which represents the reqd. storage capacity. **Ans.**

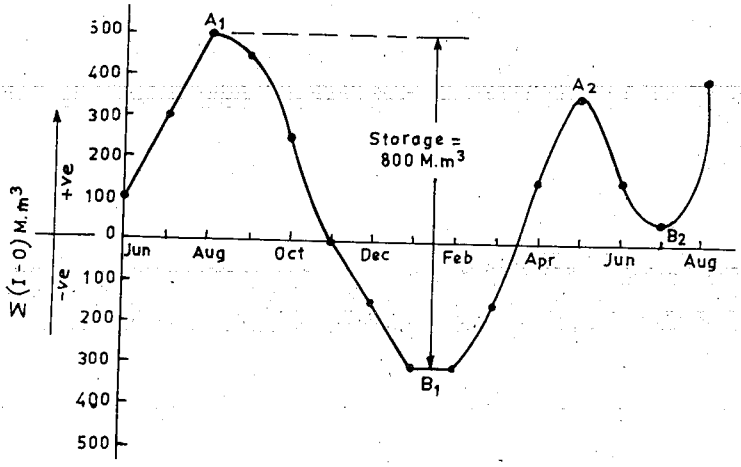


Fig. 18.16. Sequent peak algorithm for Example 18.8.

Alternatively, the values of $\Sigma (I - O)$ can be more easily computed by first computing $+ve$ and $-ve$ values of $(I - O)$, as in col (4) & (5) of Table 18.16 ; and then computing their summation values, *i.e.* values of $\Sigma (I - O)$ as in col (6) and (7) of Table 18.6. $+ve$ values represent cumulative excess inflow ; while $-ve$ values represent cumulative deficit. The maximum value out of all the values of Col (6) and (7) will represent the minimum storage required to accommodate the surplus (or to supply the deficit). This method thus, avoids the necessity of plotting peaks & troughs (curve) and the value of reqd storage capacity can be computed easily and mathematically, which procedure can even be programmed in a digital computer to help in estimation of required storage capacity. In present calculations, this peak value in col (6) & (7) is found to be **800 M.m^3. Ans.**

Table 18.16

Month	*Inflow I*	*Outflow (O)*	$(I - O)$		$\Sigma (I - O)$	
			+ ve values	*– ve values*	*Cumulative*	*Cumulative*
	M m^3	*M m^3*	*Excess (M m^3)*	*Deficit (M m^3)*	*excess (M m^3)*	*deficit (M m^3)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
June	250	150	100		100	
July	350	150	200		300	
Aug	400	200	200		500	
Sept	200	250		50		50
Oct	150	350		200		250
Nov	150	400		250		500
Dec	100	250		150		650
Jan	50	200		150		**800***
Feb	150	150	0		0	(Peak value
March	300	150	150		150	in col 6 & 7)
April	400	100	300		450	
May	450	250	200		650	
June	150	350		200		200
July	200	300		100		300
Aug	450	100	350		350	

* Max. of all the values in col (6) & (7)

Example 18.9. *Solve Example 18.7 analytically without using mass curve.*

Solution. The data given in example 18.7 is used in Table 18.17 to compute $(I - O)$ values in col (4) and (5) [– *ve i.e.* deficit values in col (4), and + *ve i.e.* Excess values in col (5)]. The values of $\Sigma (I - O)$ are finally computed in col (6) & (7). as shown :

Table 18.17

Year	*Inflow (I)* Mm^3	*Outflow (O)* Mm^3	$I - O$		$\Sigma (I - O)$	
			– ve	*+ ve*	*Cumulative*	*Cumulative*
			Deficit Mm^3	*Surplus* Mm^3	*Deficit* Mm^3	*Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1962	1642	2079	437		437	
63	2404	2079		325		325
64	2819	2079		740		1065*
65	1575	2079	504		504	
66	1596	2079	483		987	
67	2553	2079		474		474
68	1842	2079	237		237	
69	2199	2079		120		120

Out of all the values of col. (6) and (7), the max. value is 1065 Mm^3, which represents the min. storage required to accommodate this 1065 Mm^3 surplus water entering during the years 1963 and 1964. **Ans.**

Gross storage reqd. = Dead storage + Live storage

= 20% of Live storage + Live storage

= 1.2 Live storage = 1.2×1065 Mm^3 = 1278 Mm^3 **Ans.**

Example 18.10. *The yield of water in Mm^3 from a catchment area during each successive month is given in the table below :*

1.4	*2.1*	*2.8*	*8.4*	*11.9*	*11.9*
7.7	*2.8*	*2.52*	*2.24*	*1.96*	*1.68*

Determine the minimum capacity of a reservoir required to allow the above volume of water to be drawn off at a uniform rate assuming that there is no loss of water over the spillway.

Solution. The total inflow of water in 12 months

= Summation of inflow values = 57.4 Mm^3.

∴ Average monthly rate at which water is withdrawn to use the inflow fully = Av. demand rate = $57.4/12 = 4.78$ Mm^3.

Now, to draw the mass curve of inflow, the cumulative inflow values are worked out in table 18.18.

Table 18.18

Month	*Yield* (Mm^3)	*Cumulative yield* (Mm^3)
1	1.4	1.4
2	2.1	3.5
3	2.8	6.3
4	8.4	14.7
5	11.9	26.6
6	11.9	38.5
7	7.7	46.2
8	2.8	49.0
9	2.52	51.52
10	2.24	53.76
11	1.96	55.72
12	1.68	57.40

The mass inflow curve is now plotted, as shown in Fig. 18.17. A line parallel to the demand rate line is now drawn through the high point A_1 on the inflow mass curve, as shown. The maximum departure between the inflow mass curve and this line, *i.e.* B_1C_1, gives the min. storage capacity reqd. This value is read out as 20.78 Mm^3. **Ans.**

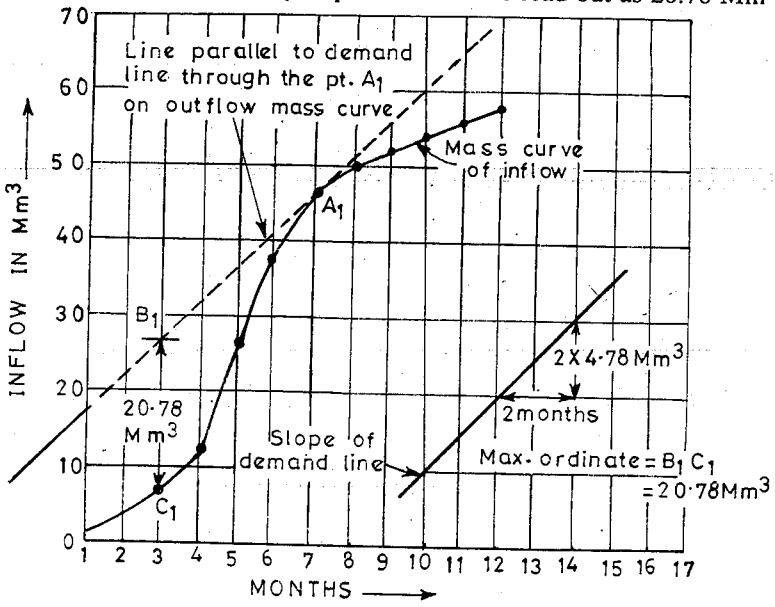


Fig. 18.17.

Analytically, the solution to the problem can be worked out, as shown in table 18.19.

Table 18.19

Month	*Inflow* Mm^3	*Outflow (Demand)* Mm^3	*Deficit* Mm^3	*Surplus* Mm^3	*Cumulative Deficit* Mm^3	*Cumulative Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1	1.4	4.78	3.38			
2	2.1	4.78	2.68			
3	2.8	4.78	1.98		8.04	
4	8.4	4.78		3.62		
5	11.9	4.78		7.12		
6	11.9	4.78		7.12		
7	7.7	4.78		2.92		20.78
8	2.8	4.78	1.98			
9	2.52	4.78	2.26			
10	2.24	4.78	2.54			
11	1.96	4.78	2.82			
12	1.68	4.78	3.10		12.70	

The highest value in col. (6) and 7 (as no spilling is allowed), is 20.78 Mm^3, which gives the min. required reservoir storage. **Ans.**

Example 18.11. *The runoff data for a river during a lean year are given below :*

Month	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*
River flow in $10^6 m^3$	140	27	35	26	16	48	212	180	116	92	67	37

What is the maximum uniform demand that can be met? What is the storage capacity required to meet this demand? What minimum initial storage is necessary? When does the reservoir become empty?

Solution. Total inflow in the year = Summation of the given monthly discharges

$= 996$ M. m^3.

Average monthly rate at which water can be withdrawn to avoid any wastage *i.e. max average monthly rate*

$$= \frac{996}{12} = 83 \text{ Mm}^3 \quad \textbf{Ans.}$$

Now, to determine storage capacity etc. we carry out the computations in table 18.20.

Table 18.20

Month end	*Monthly Inflow in reservoir in Mm^3.*	*Monthly outflow from reservoir in Mm^3.*	*Monthly deficit in reservoir (to be supplied from storage) in Mm^3*	*Monthly surplus in reservoir in Mm^3*	*Consecutive cumulative deficit in Mm^3*	*Consecutive cumulative surplus in Mm^3*	*Net water available in the reservoir in Mm^3*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
Jan	140	83		57		57	(+) 57
Feb	27	83	56				
March	35	83	48				
April	26	83	57				
May	16	83	67				
June	48	83	35		263		(–) 206
July	212	83		129			
Aug	180	83		97			
Sept	116	83		33			
Oct	92	83		9		268	(+) 62
Nov	67	83	16				
Dec	37	83	46		62		0

The highest value in col. (6) and (7) is **268 Mm^3**, which represents the minimum storage capacity required to meet the demand without any spilling. **Ans.**

To compute the min. initial storage, we compute in col. (8), the net storage left in the reservoir with the above inflows and outflows. The maximum negative storage here works out to be 206 Mm^3. In order that the reservoir fully meets the demand with the above inflows & outflows, there should be no negative storage in it, and in the limiting case the max. negative storage should be equal to zero. Hence, the min. initial storage in the reservoir should be **206 Mm^3**, which will just meet the shortage created in the reservoir by *June end,* when the reservoir will become empty. **Ans.**

Example 18.12. *The storage capacity of a reservoir for a flood control project is to be determined. The estimated cost of damage if the emergency spillway is topped is Rs. 10 lacs for each event. The interest rate is 6% and the reservoir life is 50 years. Six reservoir designs of different storage capacities ; the probabilities of exceeding those capacities in any given year, and the estimated initial cost for construction of the reservoir are as follows :*

Reservior design number	*1*	*2*	*3*	*4*	*5*	*6*
Flood storage capacity in lac cubic metres	*30*	*35*	*40*	*45*	*50*	*55*
Probability of exceeding storage capacity in any given year	*0.15*	*0.10*	*0.06*	*0.04*	*0.02*	*0.01*
Estimated initial cost, in lac rupees	*25*	*30*	*35*	*40*	*45*	*50*

Determine the optimal storage capacity of the reservoir.

(A.M.I.E. 1989)

Solution. The required computations are carried out in table 18.21 below.

Table 18.21

Reservior Design No.	*Reservior Capacity in lakh cum*	*Probability of exceeding storage capacity*	*No. of times the flood is likely to exceed the capacity in 50 yrs.*	*Damages caused due to flood exceeding capacity in Rs. Lakh = Rs. 10 lakh × Col (4)*	*Extra benefit over proposal No. 1 of lowest storage as datum; Rs. 75 lakh – col (5)*	*Imtial cost of each proposal in Rs. Lakh*	*Extra costs involved over proposal 1 as datum, in Rs. Lakh Col. 7 – Rs. 25 lakh*	*Net additional benefit in Rs. lakh; col. (6) – col. (8)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1	30	15%	7.5	75	—	25	—	—
2	35	10%	5	50	25	30	5	20
3	40	6%	3	30	45	35	10	35
4	45	4%	2	20	55	40	15	40
5	50	2%	1	10	65	45	20	45
6	55	1%	0.5	5	70	50	25	45

The above table computes the damages caused by overflooding for each proposal, the max. damage of Rs. 75 lakhs being for proposal 1 - having min. reservoir capacity; and the min. damage of Rs. 5 lakh being for proposal 6 – having max. storage capacity.

The additional benefits thus provided by proposals 2 to 6 over proposal 1 are computed in col (6) as 75 lakhs – col (5). Extra costs are also worked out in col (8).

These calculations reflect that in proposal 2, we spend extra Rs. 5 lakh (over proposal 1) and get a benefit of 25 lakhs (over proposal 1 of course) ; *i.e.* a net benefit of Rs. 20 lakh. Similarly, in proposal 3, we spend extra Rs. 10 lakhs & get a benefit of Rs. 45 lakhs over proposal 1; *i.e.* a net benefit of Rs. 35 lakhs. Similarly, we find that our net benefit increases up to proposal 5, and afterwards, as in proposal 6, it becomes equal to that in proposal 5. Hence, the increase in reservoir capacity up to proposal 5 gives us increasing net returns, but thereafter we don't get any extra net returns. Investing more, beyond proposal 5, would, therefore, not be optimal, and thus, proposal 5 can be considered as the most optimum design.

Hence, the optimum storage capacity of the reservoir will correspond to that of proposal 5, *i.e.* **50 lakh m^3. Ans.**

18.10. Estimation of Demands and Optimal Reservoir Operations

In the previous articles, it was explained as to how the reservoir capacity can be determined for satisfying a certain given downstream demand or *vice versa.* This demand may be constant or may vary throughout the year.

The demand pattern mainly depends upon the purpose for which the reservoir has been constructed. Hence, it has to be different for different types of reservoirs. Moreover, the demands are man made, and should be adjusted in such a way that the maximum benefits can be obtained with the minimum cost. If by reducing the demands slightly, a lot of expenditure can be saved, then we must go in for that. This is known as *optimal planning* for dam reservoirs, and involves economic considerations, discussed afterwards.

Demand patterns for various types of reservoirs are explained below :

(*i*) *Single Purpose Conservation Reservoir.* A single purpose conservation reservoir shall store water only to meet one particular demand, such as water supply or irrigation or hydroelectric power generation, etc. In all such cases, the demand for water can be predicted with a fair degree of accuracy for different parts of the year, and the storage capacity can be worked out, as explained earlier. The storage capacity so worked out should be otherwise feasible and should be justified on cost benefit considerations.

(*ii*) *Single Purpose Flood Control Reservoir.* In case of a single purpose flood control reservoir, there is no specific downstream demand at all, except to satisfy that the downstream release should not exceed the safe carrying capacity of the channel. Since the effect of constructing a flood control reservoir is to moderate and reduce the flood peaks by absorbing certain volume of flood, and then gradually releasing it when the flood subsides (Fig. 18.18), the capacity of such a reservoir should be sufficient to absorb this excess volume of flood.

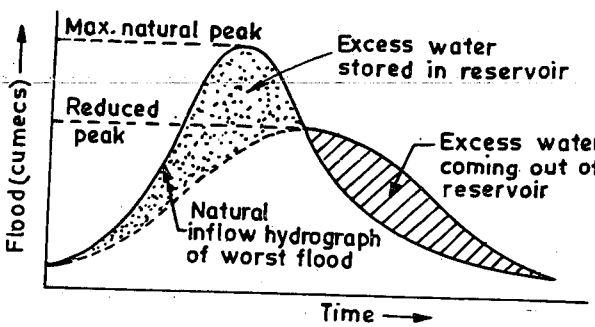


Fig. 18.18.

It, therefore, becomes evident that the capacity of such a

reservoir does not depend upon the pattern of demand but mainly depends upon the hydrograph of the worst flood that is likely to enter this reservoir and also upon the downstream permissible H.F.L. and the safe carrying capacity of the channel.

The hydrograph of the worst inflow flood can be found from the hydrological investigations, and the safe peak rate of outflow can be determined from the downstream channel conditions. The capacity of the reservoir required to moderate the inflow peak to a value equal to or less than the downstream safe peak can then be found by hit and trial method with the help of *flood routing*. Flood routing is the process by which the hydrograph of the moderated flood can be determined, and is explained a little later.

(*iii*) *Multipurpose Reservoirs.* Single purpose reservoirs are seldom constructed these days. Reservoirs are therefore, generally designed to serve more than one purpose. For example, a reservoir constructed for conserving water for irrigation may be combined with its flood control purpose. The head available may simultaneously be utilised for generating hydro-electric power. This combination of irrigation, flood control, and power is generally adopted in designing multipurpose reservoirs in India.

For proper optimal planning of such a multipurpose reservoir, a schedule of operations must be finalised from a number of tentative schedules drawn on the basis of available data and past similar experiences. The schedule which gives maximum total benefits for the various design purposes, without encroaching upon the lower or upper limits of storage, is accepted for estimation and for initial operations. This schedule may be tried and further modified on the basis of the past experience and future needs.

A multipurpose reservoir constructed on a perennial snow fed river in North India (such as Bhakra reservoir) can be operated on the following lines :

Fig. 18.19 shows the typical schedule of operations for such a reservoir. The reservoir water will normally fluctuate between minimum pool level and normal pool level (*i.e.* maximum conservation level) for satisfying irrigation needs. The minimum pool level will ensure the generation of firm power, as the water level shall not normally

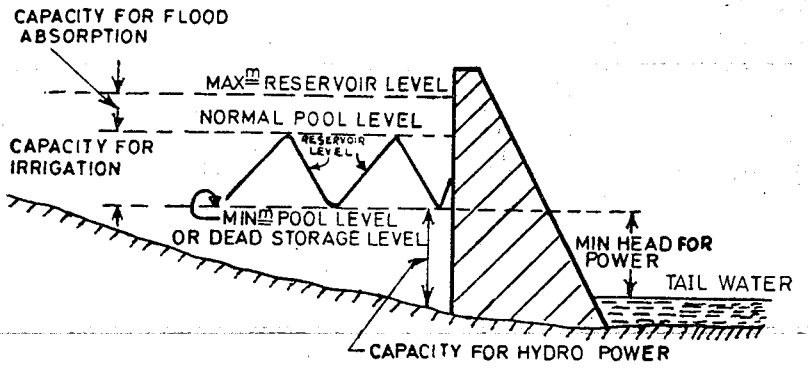


Fig. 18.19.

be allowed to go below the minimum pool level. Any serious flood which may occur after the reservoir is full, shall be absorbed between the normal pool level and the miximum pool level, at all times. As soon as the water level reaches the normal pool level, the spillway will start functioning. But the rate of water discharged over the spillway shall be in accordance with the downstream safe carrying capacity of the

channel, and hence, may be less than the rate of inflow. The water shall, then, be stored temporarily between the normal pool level and the maximum reservoir level, and discharged at a safe rate. As the flood subsides, and the excess water goes out slowly, the water level will again fall up to the normal pool level, and the reservoir will be ready to absorb another flood.

For irrigation, we generally require water from April to June for Kharif crops, and from middle of November to middle of February for Rabi crops. There are heavy rains during monsoon period of say June to middle of October. At the start of the monsoon season, the reservoir is quite depleted because of heavy outflow for Kharif crops and constant release for power generation. Thus, just before the monsoon, a large reservoir capacity is available, which not only conserves the water but also serves to control floods that may occur at this time. During the monsoon, the irrigation demand will almost be nil and water shall be released only for power generation. The reservoir level will, therefore, go on increasing steadily and will be allowed to go up to the normal pool level by the end of normal monsoon season, say middle of October. Any untimely flood occurring after the reservoir is full up to normal pool level, shall be absorbed between normal pool level and the maximum reservoir level, as explained earlier.

After the monsoon is over, the inflow is considerably reduced (almost nil) and water is constantly drawn for Rabi crops and power generation. Due to this, the reservoir level goes on falling up to about middle of February, when it is usually the lowest, somewhere near the minimum pool level (*i.e.* dead storage level). At this time, snow starts melting in the head reaches of the river, which augments the river discharge. Some winter cyclonic rains during December and January may also be helpful in augmenting the reservoir storage. From February-March onwards, there are large supplies due to melting of snow, and an equal burden is to be absorbed by the reservoir for satisfying the Kharif irrigation withdrawals. Thus, during April, May and June, the reservoir level may rise or fall slightly at slow rate; and at the start of monsoon, *i.e.* at the end of June or beginning of July, the reservoir will be quite depleted and prepared for full replenishment of its supplies.

FLOOD ROUTING OR FLOOD ABSORPTION

The hydrograph of a flood entering a reservoir, will change in shape as it emerges out of the reservoir, because certain volume of its water is stored in the reservoir temporarily and is let off as the flood subsides. The base of the hydrograph, therefore, gets broadened, its peak gets reduced, and, of course, the time of peak is delayed. The extent by which the inflow hydrograph gets modified due to the reservoir storage can be computed by a process known as *flood routing,* and more particularly as *reservoir routing* (to differentiate it from routing though 'river channels').

Since the flood protection reservoirs are generally located many km upstream of the cities which are to be saved against floods, it is sometimes necessary to route the outflow hydrograph of the reservoirs up to these downstream localities. The reservoir outflow hydrograph may then be routed through this much length of river channel, so as to obtain the final shape of the hydrograph at the affected cities. This routing, in which the stream itself acts like an elongated reservoir, is known as *channel routing* or *river routing.*

A variety of routing methods are available, and they can be broadly classified into two categories, *viz*

(*i*) *hydrologic routing ; and*

(*ii*) *hydraulic routing*

Hydrologic routing methods employ essentially the *equation of continuity ; whereas the hydraulic routing methods employ the continuity equation together with the equation of motion of unsteady flow.* The basic differential equations used in hydraulic routing are popularly known as *St. Venant equations*, and afford a better description of the unsteady flow than the hydrologic methods. We shall, however, cofine ourselves in the book to the hydrologic methods only.

18.11. Hydrologic Reservoir Routing Methods

The passage of a flood wave through a reservoir or a river reach is an *unsteady flow* phenomenon. In hydraulics, we classify it as a *gradually varied flow.* The *equation of continuity* used in all the hydrologic routing methods, as the primary equation, states that the difference between the inflow and the outflow rate is equal to the rate of change of storage ; *i.e.*

$$I - O = \frac{dS}{dt} \qquad ...(18.9)$$

where I = Inflow rate
O = Outflow rate
S = Storage

Alternatively, in a small time interval Δt, the difference between the total inflow volume and the total outflow volume is equal to the change in storage volume ; *viz.*

$$\underset{\substack{\text{(Inflow volume)}\\ I(v)}}{\bar{I}\,\Delta t} - \underset{\substack{\text{(Outflow vol.)}\\ O(v)}}{\bar{O}\,\Delta t} = \Delta S \qquad ...(18.10)$$

where $\bar{I}$ = Average inflow (rate) in time Δt
$\bar{O}$ = Average outflow (rate) in time Δt
ΔS = change in storage during the time Δt

Since $\bar{I} = \frac{I_1 + I_2}{2}$;

$\bar{O} = \frac{O_1 + O_2}{2}$;

$\Delta S = S_2 - S_1$;

where suffixes 1 and 2 denote the beginning and the end of the time interval Δt.

Eq. (18.10) can then be written as :

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left(\frac{O_1 + O_2}{2}\right)\Delta t = S_2 - S_1 \qquad ...(18.11)$$

The time interval Δt should be sufficiently short, so that the inflow and outflow hydrographs can be assumed to be in straight line, in that time interval. Moreover, Δt must be shorter than the time of transit of the flood wave through the reservoir or the given river reach.

The above relationship seems to be very simple, but its evaluation is not easily possible without drastic simplifying assumptions. This is because of the fact that the relations between time and rate of inflow, elevation and storage of reservoir, and

elevation and rate of outflow, cannot be expressed by simple algebraic equations. They are, respectively represented by the *inflow flood hydrograph, the elevation-storage curve, and the outflow-elevation curve.* The first two curves obviously cannot be represented by any simple equations, and the third may be represented by the spillway-discharge equation ($Q = 1.71\ LH^{3/2}$) only if, the discharge through the outlets is neglected. If the discharge though the outlets is also not neglected, then all the three curves will be unamenable to simple mathematical treatment, without drastic assumptions.

Several procedures have, however, been suggested by different investigators to solve the above basic equation (Eq. 18.11) by rearranging the components in different manners. Depending upon the different procedures adopted for solving the above basic equation, the following hydrologic methods may be used for reservoir routing :

(1) *Trial and Error method*

(2) *Modified Pul's method or Storage indication method ; and*

(3) *Goodrich method.*

The first method is discussed here, while the detailed description of other methods is available in authors another book titled "*Hydrology and Water Resources Engineering*" and may be referred to in specific needs.

18.11.1. Trial and Error Method of Reservoir Routing. Trial and Error method is widely adopted with the assistance of computers to reduce the time taken in long calculations involved in this method. This method arranges the basic routing equation (Eq. 18.11), as follows :

$$\frac{I_1 + I_2}{2} \cdot \Delta t = \frac{O_1 + O_2}{2} \cdot \Delta t + (S_2 - S_1) \qquad ...(18.11\ a)$$

The procedure involves assuming of a particular level in the reservoir at the end of the interval Δt, and computing the values on the right side of the above Eq. (18.11 *a*). The summation of $\frac{O_1 + O_2}{2} \cdot \Delta t$ and $(S_2 - S_1)$ is then compared with the known value of $\frac{I_1 + I_2}{2} \cdot \Delta t$. If the two values tally, then the assumed reservoir elevation at the end of the interval is supposed to be O.K. ; otherwise this is changed, and the process is repeated till the required matching is obtained.

This method gives quite reliable results, provided the chosen time interval (Δt) is sufficiently small, so that the mean of the outflow rates at the start and the end of the given interval may be taken as the average throughout the interval.

Procedure : The following detailed procedure may be adopted in this method, to complete the involved computations :

Data to be given : (*i*) the inflow hydrograph

(*ii*) Elevation capacity curve or Elevation area curve

(*iii*) Elevation outflow curve.

Steps involved in computations

(*i*) Divide the inflow flood hydrograph into a number of small intervals. The time interval should be so chosen, as not to miss the peak values.

(*ii*) Fix the normal pool level at which the spillway crest is provided, and the level at which the flood enters the reservoir ; the two are generally taken to be the same, as

it is assumed that this worst design flood enters the reservoir only after the reservoir is full up to the normal pool level.

(*iii*) Work out the spillway and the outlet discharge rating curves, if not given.

(*iv*) Work out the elevation-capacity curve for the reservoir from the elevation-area curve, if the former is not given, using cone formula, *i.e.* $V = \Sigma \frac{h}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$, where h is the contour interval. (18.12)

(*v*) Start with the first interval and compute the total inflow during the interval by multiplying the average inflow rate at the beginning and the end of the interval, with the period of the interval.

$$\therefore \qquad I_{(V)} = \frac{I_1 + I_2}{2} (\Delta t)$$

where I_1 = Inflow rate at the start of the interval
I_2 = Inflow rate at the end of the interval
Δt = Duration of the interval
$I_{(V)}$ = Total inflow volume during the interval.

(*vi*) The reservoir level at the start of the flood (*i.e.* start of first interval) is known. Assume a trial value for the reservoir level at the end of the interval.

(*vii*) compute the total outflow during the interval

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot \Delta t$$

where O_1 = Outflow rate at the start of the interval, corresponding to the given reservoir level.
O_2 = Outflow rate at the end of the interval, corresponding to the assumed reservoir level.
Δt = Duration of the interval
$O_{(V)}$ = Total outflow volume during the interval.

(*viii*) Using the elevation-storage curve for the reservoir, determine the storage S_1 and S_2 at the beginning and the end of the interval, corresponding to the known and the assumed reservoir levels, respectively. Their difference $S_2 - S_1 = \Delta S$, represent the amount of flood stored in the reservoir during the interval.

(*ix*) Add the volume of outflow $O_{(V)}$ obtained in step (*vii*) to the values of ΔS obtained in step (*viii*), and compare it with the inflow volume $I_{(V)}$, calculated in step (*v*). The two values must be equal (*i.e.* $I_{(V)} = O_{(V)} + \Delta S$). If this is so, the assumed reservoir level is correct, otherwise, change it and repeat the procedure till this coincidence is obtained.

(*x*) All the above steps should be repeated for other time intervals, till the entire flood is routed or still further, till the reservoir level returns to pre-flood pool level.

(*xi*) Outflow ordinates are plotted so as to obtain the outflow hydrograph. The point at which it crosses the inflow hydrograph gives the peak outflow rate. From this time, the rate of outflow begins to fall due to decrease in the inflow rate.

(*xii*) The time lag between the two peaks is evaluated as to give the time lag.

An example has been solved to make the procedure very clear.

Example 18.13. *The inflow flood discharges for a possible worst flood are tabulated in Table 18.22 at suitable intervals starting from 0.00 hours on august 20, 1975.*

Table 18.22

Time from start in hr	*0*	*6*	*12*	*18*	*24*	*30*	*36*	*42*	*48*	*51*	*60*	*66*	*78*	*90*	*102*	*114*
Discharge in cumecs	*0*	*50*	*280*	*610*	*1290*	*1900*	*2130*	*1900*	*1600*	*1440*	*1060*	*780*	*500*	*370*	*220*	*130*

This flood approaches a reservoir with an uncontrolled spillway, the crest of which is kept at RL 140.0 m. Determine the maximum reservoir level and the hydrograph of the routed flood. Values of reservoir capacity (above spillway crest) and outflow discharge at various elevations are tabulated in Tables 18.23 and 18.24 respectively.

Table 18.23

Elevation in metres	*140.0*	*141.0*	*142.0*	*143.0*	*144.0*	*145.0*	*146.0*
Reservoir storage with spillway crest as datum in million cubic metres (m.c.m.)	*0.0*	*15.0*	*35.0*	*60.0*	*95.0*	*140.0*	*240.0*

Table 18.24

Elevation	*Outflow discharge in cumecs*
140.0	*0*
141.0	*170*
142.0	*482*
143.0	*883*
144.0	*1,360*
145.0	*1,905*
146.0	*2,500*

Solution. The elevation storage curve and the elevation outflow curve are plotted with the help of Tables 18.23 and 18.24, as shown in Fig. 18.20 and 18.21 respectively. The hydrograph of the given flood is plotted in Fig. 18.22. Flood routing is carried out by hit and trial method as shown in Table 18.25 and as explained earlier. This table is otherwise self-explanatory.

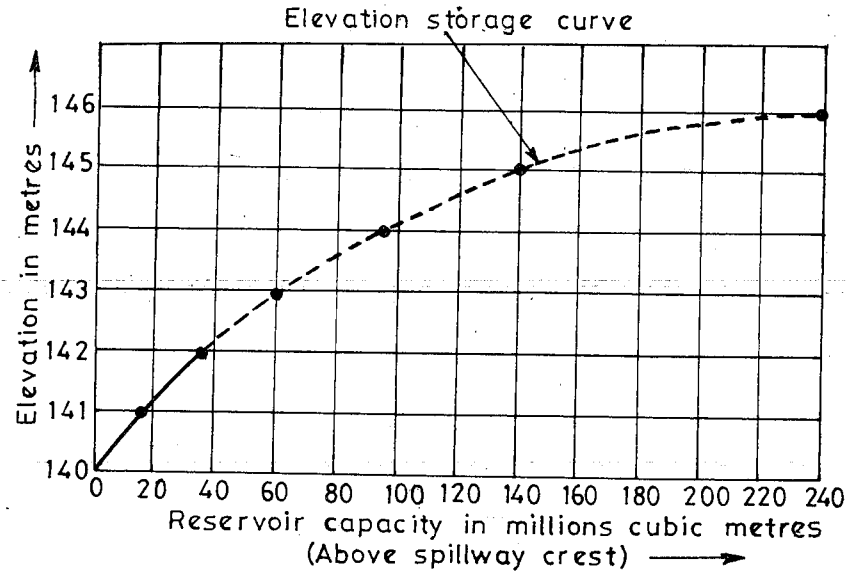


Fig. 18.20. Elevation Storage curve for example 18.13.

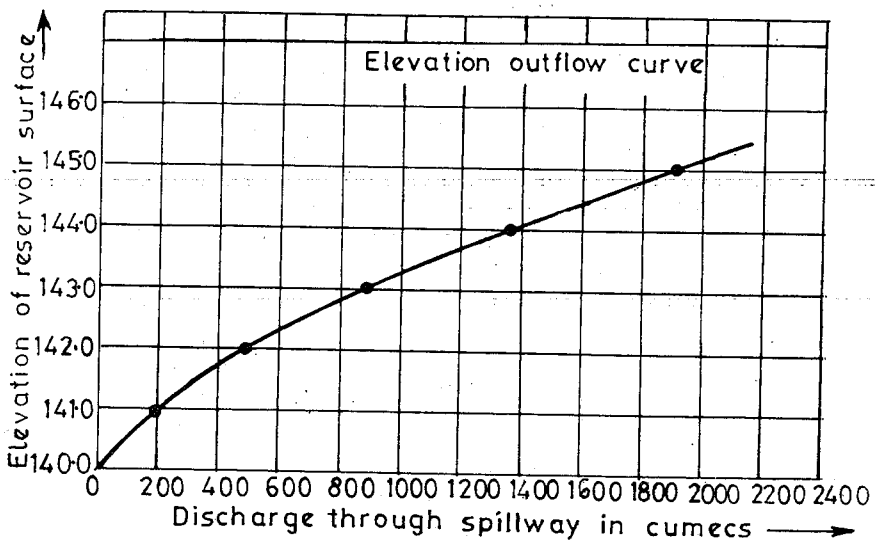


Fig. 18.21. Elevation-Outflow curve for example 18.13.

The outflow hydrograph is plotted from col. (7) of table 18.25, as shown in Fig. 18.22 by the dotted curve. The point at which it intersects the inflow hydrograph, represents the peak of the routed flood.

The peak of outflow flood works out to be **1458 cumecs,** while the peak of the inflow flood was 2,130 cumecs. The time lag is found to be **15 hours** (from Fig. 18.22). The maximum reservoir level is found from Table 18.25, to be **144.18 metres. Ans.**

Table 18.25

Time from start		Time interval (Δt)	Reservoir elevation at the beginning of interval	Inflow at the start of interval (I_1)	Inflow at the end of interval (I_2)	Volume of inflow during the interval $I(V) = (I_1 + I_2)/2$	Outflow at the start of interval (O_1)	Trial reservoir elevation at the end of interval	Outflow at the end of interval (O_2)	Mean outflow volume $O(V) = [(O_1 + O_2)/2]\,\Delta t$	Storage capacity at start of interval (S_1)	Storage capacity at the end of interval (S_2)	Change in storage during the interval $(\Delta S) = (S_2 - S_1)$	Outflow volume + change in storage; i.e. $O_{(V)} + \Delta S$	Compare $O_{(V)} + \Delta S$ with $I_{(V)}$, i.e. col. (14) with col. (6) and Remarks
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Hrs.		Hrs.	m	I_1 m^3/s	I_2 m^3/s	I $M.m^3$	O_1 m^3/s	m	O_2 m^3/s	$O_{(v)}$ $M.m^3$	S_1 reckoned over spillway crest $M.m^3$	S_2 M.m3	$(S_2 - S_1)$ *i.e.* (ΔS) $M.m^3$	$O_{(V)} + \Delta S$ $M.m^3$	Whether $(O_{(V)} + \Delta S)$ is large or small as compared to $I_{(V)}$
0—6	6		140.0	0.0	50	0.54	0.0	140.1	5.67	0.06	0.0	1.2	1.2	1.26	Large and hence change the trial value
								140.03	0.88	0.01		0.525	0.525	0.535	O.K.

...contd

Table 18.25 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
6-12	6	140.03	50	280	3.56	0.88	140.3	28.7	0.31	0.525	4.0	3.475	3.785	large
							140.25	21.2	0.23	"	3.85	3.325	3.555	O.K.
12-18	6	140.25	280	610	9.62	21.2	140.5	60.0	0.88	3.85	8.0	4.15	5.03	small
							141.0	170.0	2.06	"	15.0	11.15	13.21	large
							140.8	121.6	1.54	"	13.0	9.15	10.69	large
							140.75	110.2	1.42	"	12.05	8.20	9.62	O.K.
18-24	6	140.75	610	1290	20.5	110.2	141.5	312.2	4.57	12.05	26.0	13.95	18.52	small
							141.6	343.0	4.89	"	27.7	15.65	20.54	O.K.
24-30	6	141.06	1290	1900	34.4	343.0	142.5	672	10.95	27.7	47.0	19.3	30.25	small
							142.6	713	11.4	"	50.0	22.3	33.7	small
							142.63	725	11.52	"	50.5	22.8	34.32	O.K.
30-36	6	142.63	1900	2130	43.6	725	143.5	1113	19.85	50.5	76.0	25.5	45.35	large
							143.4	1065	19.23	"	73.0	22.5	41.73	small
							143.45	1087	19.55	"	74.5	24.0	43.55	O.K.
36-42	6	143.45	2130	1900	43.6	1087	144.0	1360	26.44	74.5	95.0	20.5	46.94	large
							143.95	1335	26.16	"	92.0	17.5	43.65	O.K.
42-48	6	143.95	1900	1600	37.8	1335	144.5	1620	13.9	92.0	115.0	23.0	64.9	large
							144.2	1461	30.2	"	102.0	10.0	40.2	large
							144.15	1435	29.9	"	100.0	8.0	37.9	O.K.

...contd

Table 18.25 (Contd.)

(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*	*(14)*	(15)
48-51	3	144.15	1600	1440	16.4	1435	114.18	1458*	15.5	100.0	101.0	1.0	16.5	O.K.
51-60	9	144.18*	1440	1060	40.5	1458*	144.10	1410	46.5	101.0	98.0	–3.0	43.5	large
							144.05	1385	46.1	"	95.5	–5.0	41.1	O.K.
60-66	6	144.05	1060	780	19.9	1385	143.8	1258	28.5	95.5	87.0	–8.5	20.0	O.K.
66-78	12	143.80	780	500	27.8	1258	142.2	974	48.2	87.0	67.0	–20.0	28.2	O.K.
78-90	12	143.20	500	370	18.8	974	142.60	713	35.4	67.0	50.0	–17.0	18.4	O.K.
90-102	12	142.60	370	220	12.7	713	142.0	482	25.8	50.0	35.0	–15.0	10.8	small
							142.08	510	26.5		36.2	–13.8	12.7	O.K.

*Results : Peak rate of outflow = 1,458 cumecs
Maximum reservoir level = 144.18 metres
Time lag = 15 hours

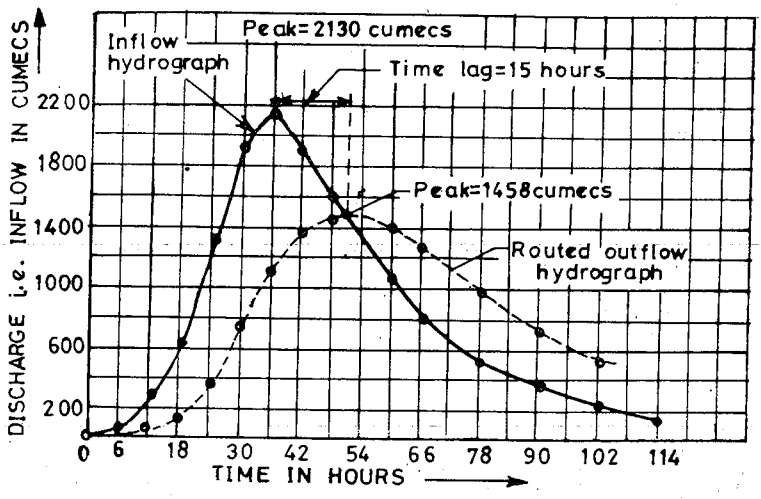


Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

Example 18.14. *The hydrograph of inflow to a reservoir is given in the table below.*

Time(days)	*0*	*2*	*4*	*6*	*8*	*10*	*12*	*14*	*16*	*18*	*20*
Flow (m^3/s)	*60*	*115*	*425*	*550*	*440*	*320*	*260*	*200*	*150*	*110*	*60*

The reservoir is full at the start of the flood inflow. The storage S of reservoir above the spillway crest, is given in million cubic metres by : $S = 8.64\,h$, where h is the head in metres above the crest. The discharge over the spillway is given in cumecs by $Q = 60\,h$. Find the head over the spillway crest at the end of 8th day of the flood.

Solution. Let h_1 be the head over the spillway crest at the end of 2 days ; h_2 at the end of 4 days ; h_3 at the end of 6 days ; and h_4 at the end of 8 days. Now we will consider the position, interval by interval.

(*i*) **0—2 day interval**

Inflow $I_1 = 60$ cumecs.

$I_2 = 115$ cumecs.

$$\therefore \quad I_{(V)} = \left(\frac{I_1 + I_2}{2}\right) t = \left(\frac{60 + 115}{2}\right) 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 15.12\ \text{M.m}^3.$$

Outflow $O_1 = 0$

$O_2 = 60\,h_1$

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot t = \left(\frac{60h_1}{2}\right) \times 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 5.184\,h_1\ \text{M.m}^3.$$

Storage $S_1 = 0$

$S_2 = 8.64\,h_1\ \text{M.m}^3$

$\therefore \quad \Delta S = 8.64\,h_1$

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Now using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$15.12 = 5.184\, h_1 + 8.64\, h_1 \quad \text{or} \quad 15.12 = 13.824\, h_1$$

$$\therefore \quad h_1 = \frac{15.12}{13.824} = \mathbf{1.093\ m.}$$

(*ii*) **2—4 day interval**

Inflow $I_1 = 115$ cumecs ; $I_2 = 425$ cumecs

$$I_{(V)} = \left(\frac{115 + 425}{2}\right) 2 \times 86400\ \text{m}^3 = 46.656\ \text{Mm}^3.$$

Outflow $O_1 = 60\, h_1$; $O_2 = 60\, h_2$

$$O_{(V)} = \left(\frac{60h_1 + 60h_2}{2}\right) 2 \times (86400)\ \text{m}^3 = (h_1 + h_2) \times 5.184\ \text{Mm}^3.$$

Storage $S_1 = 8.64\, h_1$; $S_2 = 8.64\, h_2$

$$\Delta S = 8.64\, (h_2 - h_1)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$46.656 = (h_1 + h_2)\ 5.184 + 8.64\, (h_2 - h_1)$$

Using $h_1 = 1.093$ m, we get

$$46.656 = 5.666 + \mathbf{5.184\, h_2} + \mathbf{8.64\, h_2} - 9.443$$

or $$50.433 = 13.824\, h_2 \quad \text{or} \quad \mathbf{h_2 = 3.648\ m.}$$

(*iii*) **4—6 day interval**

Inflow $I_1 = 425$ cumecs ; $I_2 = 550$ cumecs

$$\therefore \quad I_{(V)} = \frac{425 + 550}{2} \times 2 \times 86400\ \text{m}^3 = 84.24\ \text{M.m}^3.$$

Outflow $O_1 = 60h_2$; $O_2 = 60h_3$

$$O_{(V)} = \frac{60h_2 + 60h_3}{2} \times 2 \times 86400\ \text{m}^3 = (h_2 + h_3)\ 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64h_2$; $S_2 = 8.64h_3$

$$\Delta S = 8.64\, (h_3 - h_2)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$84.24 = (h_2 + h_3)\ 5.184 + 8.64\, (h_3 - h_2)$$

Using $h_2 = 3.648$ m in this eqn., we get

$$84.24 = 18.911 + \mathbf{5.184\ h_3} + \mathbf{8.64\, h_3} - 31.518$$

$$\therefore \quad 96.848 = 13.824\, h_3 \quad \therefore \quad \mathbf{h_3 = 7.00\ m.}$$

(*iv*) **6–8 day interval.**

Inflow $I_1 = 550$ cumecs ; $I_2 = 440$ cumecs

$$I_{(V)} = \left(\frac{550 + 440}{2}\right) 2\ (86400)\ \text{M.m}^3 = 85.536\ \text{M.m}^3.$$

Outflow $O_1 = 60\, h_3$; $O_2 = 60\, h_4$

$$O_{(V)} = \frac{60h_3 + 60h_4}{2} \times 2 \times 86400\ \text{m}^3 = (h_3 + h_4)\ 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64\, h_3$; $S_2 = 8.64\, h_4$

$$\therefore \quad \Delta S = S_2 - S_1 = 8.64\, (h_4 - h_3)$$

Using $h_3 = 7.00$ m in this eqn., we get

$$85.536 = 36.288 + \mathbf{5.184\, h_4} + \mathbf{8.64\, h_4} - 60.48$$

or $109.728 = 13.824\, h_4$ or $\mathbf{h_4 = 7.94\ m.}$

Hence, the head over the spillway crest at the end of the 8th day = 7.94 m. **Ans.**

Example 18.15. *A small reservoir has an area of 5000 hectares* ($50 \times 10^6\ m^2$) *at spillway crest level. Banks are essentially vertical above the spillway crest level. The spillway is 50 m long and has a coefficient C of 2.2 in equation* $q = CH^{3/2}$. *The inflow to the reservoir is given in the table below :*

Time from start (hrs.)	*0*	*6*	*12*	*24*	*30*	*36*	*42*	*48*	*54*
Inflow (m^3/s)	*40*	*340*	*722*	*320*	*192*	*118*	*80*	*56*	*40*

Compute the maximum outflow discharge over the spillway; and the reservoir level to be expected if the reservoir level was at the spillway crest at the start.

(Bhopal University, 1980)

Solution. As done in example 18.11, let us assume that the head over the spillway crest at the end of the various intervals be h_1, h_2, h_3, h_4 metres. Then

For interval 0—6 hr.

Inflow $I_1 = 40$ cumecs ; $I_2 = 340$ cumecs

$$I_{(V)} = \left(\frac{40 + 340}{2}\right)(6 \times 60 \times 60)\ m^3 = 4.104\ M.m^3.$$

Outflow $O_1 = 0$

$$O_2 = 2.2 \times 50 \times h_1^{3/2} = 110\, h_1^{3/2} \qquad [\because Q \text{ over spillway} = C.L.H.^{3/2}]$$

$$O_{(V)} = \left(\frac{O_1 + O_2}{2}\right) \cdot t = \left(\frac{0 + 110 h_1^{3/2}}{2}\right)(6 \times 60 \times 60)\ m^3 = 1.19\, h_1^{3/2}\ M.m^3.$$

Storage $S_1 = 0$

$$S_2 = (50 \times 10^6)\, h_1\ m^3. \qquad [\because \text{Storage above the spillway is rectangular} = \text{Area of the reservoir} \times \text{ht. of water}]$$

$$\therefore \quad \Delta S = 50\, h_1\ M.m^3.$$

Now, using $I_{(V)} = O_{(V)} + \Delta S$, we have

$$4.104 = 1.19\, h_1^{3/2} + 50 h_1 \qquad \text{or} \qquad h_1^{3/2} + 42 h_1 - 3.45 = 0$$

Solving by hit and trial, we get

$$\mathbf{h_1 = 0.082\ m.}$$

For interval 6–12 hr.

$$I_{(V)} = \left(\frac{340 + 722}{2}\right)(6 \times 60 \times 60)\ m^3 = 11.47\ M.m^3$$

$$O_{(V)} = \frac{110\, h_1^{3/2} + 110\, h_2^{3/2}}{2} \cdot (6 \times 60 \times 60)\ m^3 = (h_1^{3/2} + h_2^{3/2})\ 1.19\ M.m^3$$

$$\Delta S = 50\,(h_2 - h_1)\ M.m^3$$

$$\therefore \quad 11.47 = 1.19\,(h_1^{3/2} + h_2^{3/2}) + 50(h_2 - h_1)$$

Using $h_1 = 0.082$, we have

$$9.65 = 0.02 + h_2^{3/2} + 42h_2 - 3.44 \quad \text{or} \quad 13.07 = h_2^{3/2} + 42h_2,$$

or $h_2^{3/2} + 42h_2 - 13.07 = 0$ $\therefore$ $\mathbf{h_2 = 0.31\ m.}$

For interval 12—24 hr.

Inflow $I_{(V)} = \left(\frac{722 + 320}{2}\right)(12 \times 60 \times 60)\ \text{m}^3 = 22.51\ \text{M.m}^3$

Outflow $O_1 = 110\ h_2^{3/2}$; $O_2 = 110\ h_3^{3/2}$

$$O_{(V)} = \frac{110\ (h_2^{3/2} + h_3^{3/2})}{2} \times (12 \times 60 \times 60)\ \text{m}^3 = 2.38\ (h_2^{3/2} + h_3^{3/2})\ \text{M.m}^3$$

Storage $S_1 = 50\ h_2$ $S_2 = 50\ h_3$

$$\Delta S = 50\ (h_3 - h_2)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$\therefore$ $22.51 = 2.38\ (h_2^{3/2} + h_3^{3/2}) + 50\ (h_3 - h_2)$

Using $h_2 = 0.31$ m, we have

$$9.46 = 0.17 + h_3^{3/2} + 12.2\ h_3 - 3.78$$

$$13.07 = h_3^{3/2} + 12.2\ h_3.$$

or $h_3^{3/2} + 12.2\ h_3 - 13.07 = 0$ $\therefore$ $\mathbf{h_3 = 0.99\ m.}$

For interval 24—30 hr.

Inflow $I_{(V)} = \left(\frac{320 + 192}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 5.53\ \text{M.m}^3$

Outflow $O_{(V)} = \frac{110\ (h_3^{3/2} + h_4^{3/2})}{2}\ (6 \times 60 \times 60) = 1.19\ (h_3^{3/2} + h_4^{3/2})$

$$\Delta S = 50\ (h_4 - h_3)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$\therefore$ $5.53 = 1.19\ (h_3^{3/2} + h_4^{3/2}) + 50\ (h_4 - h_3)$

or $4.65 = (h_3^{3/2} + h_4^{3/2}) + 9.04\ (h_4 - h_3)$

Using $h_3 = 0.99$. we have

$$4.65 = 0.99 + h_4^{3/2} + 9.04\ h_4 - 8.95$$

or $h_4^{3/2} + 9.04 h_4 - 12.61 = 0$ $\therefore$ $\mathbf{h_4 = 1.24\ m.}$

For interval 30—36 hr.

$$I_{(V)} = \left(\frac{192 + 118}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 3.35\ \text{M.m}^3.$$

$$O_{(V)} = 1.19\ (h_4^{3/2} + h_5^{3/2})$$

$$\Delta S = 50\ (h_5 - h_4)$$

$\therefore$ $3.35 = 1.19\ (h_4^{3/2} + h_5^{3/2}) + 50\ (h_5 - h_4)$

or $2.81 = (h_4^{3/2} + h_5^{3/2}) + 42\,(h_5 - h_4)$

Using $h_4 = 1.24$ m, we get

$$2.81 = 1.38 + h_5^{3/2} + 42\,h_5 - 52.08$$

or $h_5^{3/2} + 42h_5 - 50.65 = 0 \qquad \therefore \qquad h_5 = 1.17$ m

Since $h_5 < h_4$, it means that the head has started reducing, reaching its max. value of **1.24 m.**

$\therefore$ max. outflow discharge over the spillway

$$= 110\,(1.24)^{3/2} \text{ cumecs.} = 151.8 \text{ cumecs.} \quad \textbf{Ans.}$$

and, the max. reservoir level will be 1.24 m higher than the spillway crest level, which cannot be worked out as the spillway crest level is not given. **Ans.**

RESERVOIR REGULATION

18.12. Rule Curves and Operating Tables for Reservoirs

Multipurpose reservoirs need to be operated and regulated efficiently with a high degree of intelligence, intuition, and experience, in order to ensure that *they are neither left partially empty by the end of the rainy season, nor they are found full at the time of arrival of a series of peak floods, leading to heavy releases, causing floods in the downstream valley.*

Reservoir regulation committees, consisting of experts are, therefore, generally constituted to ensure issue of proper and timely directives to the staff operating the gated openings of the reservoir, to avoid any bad and inefficient operations.

Guiding tables and curves, called **rule curves** or **guide curves,** are drawn in advance, and kept ready for use for the efficient regulation of the reservoir waters, with time. Such guiding curves are normally required only for the flood season, because for the rest of the year, the reservoir will only discharge water for irrigation and hydel needs. Sometimes, **reservoir regulation manuals** are also prepared and made available to the officers, dealing with the operation of the gates of the reservoir. Such manuals provide guidelines for gate operations, and for the overall maintenance and upkeep of the dam and the reservoir.

A rule curve, either in tabular or graphical form, as framed by the experts on the basis of the past data, broadly reflects the maximum reservoir levels to be achieved by the different dates of the rainy season. *Such a curve, therefore, reflects the vacant space to be left in the reservoir on different dates or weeks of the rainy season.* Such guides may have to be revised from time to time, based on their performance during their actual use.

A typical set of such rule curves (tabular form) being followed for the operation of *Hirakud dam reservoir** are shown in table 18.26 (*a*) and (*b*).

* Hirakud dam is constructed on Mahanadi river is Orissa State of India. It is 3 mile long composite dam, with max. height of 200 ft. Spillway capacity at FRL of 630′ is 14.83 lakh cusecs. It intercepts a catchment of 32,200 sq. miles, and the catchment below the dam and up to the head of delta, i.e. Naraj, is about 18,000 sq. miles. There is no flood reserve or conservation reserve as such, and it is a general use reservoir. The river capacity at Naraj is about 9 lakh cusecs.

Table 18.26 (*a*) Rule curve for filling the Hirakud reservoir for conservation

Date of the year	*Permissible reservoir water level (Dead storage RL=590ft)*
11th August (on and upto)	600′
21st August	605′
1st September	617′
11th September	623′
21st September	627′
1st October (on and after)	630′ (F.R.L.)

Table 18.26 (*b*) Rule curve for flood releases (under ordinary circumstances) for Hirakud dam reservoir

Range of levels RL in ft	*Permissible outflow in lakh cusecs*	*Safe depletion rate in lakh cusecs*
600 — 610	2.5	4.0
610 — 615	3.0	
615 — 620	4.0	
620 — 625	4.5	
625 — 628	5.5	
628 — 630	6.5	

Prior to the above rules, Hirakud dam reservoir was following another set of rules, as given in table 18.27, which proved to be inadequate, leading to insufficient storage at the end of flood seasons for some of the years, and heavy discharges from the reservoir synchronising with floods in the lower catchment in some other years.

Table 18.27. Old Rules, specified in the Manual for Operation of Hirakud reservoir, which have now been superceded by rules of tables 18.21 (*a*) and (*b*)

S. No. of Rule	*Rule*
Rule 1	There should be no full impounding till 1st of September in normal years. The entire inflow should be let out ; care being exercised that the combined discharge of reservoir outflow and the runoff from the catchment below the dam does not exceed 10 lakh cusecs at Naraj. Generally, the inflow up to 5 lakh cusecs can be passed without hesitation. When the inflow is more than 5 lakh cusecs, then the hydrometeorological conditions in the lower catchment should be carefully studied and outflow determined.
Rule 2	Safety of dam should be the prime consideration on and at every occasion when floods are absorbed in the reservoir for purposes of moderating and regulating outflow in the reservoir. A level of + 625.0′ should be deemed to be the safe level normally for this purpose.

Another typical rule curve (graphical form), as drawn and used for Maithon dam reservoir, under *D.V.C. System**, is shown in Fig. 18.23. This operation schedule shows that the reservoir, which will be quite depleted by the start of June, will be filled up fully up to *Monsoon storage level* (RL 480′), latest by 3rd week of September, and will be maintained full, till the end of November. The reservoir will be allowed to be depleted only from December onward.

Any flood, which occurs during the last week of September, October and November, will thus, find the reservoir full, and will have to be accommodated within the *'flood reserve', i.e.* between monsoon storage level (RL 480′) and the max. permissible operational level of gates (RL 495′).

Depending upon the intensity of the coming and likely floods after the reservoir is full, the outflows will be regulated and so manipulated that the discharge from the reservoir together with the discharge contributed by the downstream area, does not exceed the safe carrying capacity of the river farther downstream.

As and when a late flood enters the reservoir, the reservoir-gate-operators may adjust the outflow to a lower value in the beginning, but may have to increase the outflow, as the flood waves continue and the weather forecasts for continued rains are received. In this operation, a stage may reach when the outflow may become quite high, causing floods in the down valley.

While manipulating these outflows, chances of synchronisation of the discharge of the downstream uncontrolled catchment will also have to be considered properly.

Say for example, if we release 1 lakh cusecs discharge from the reservoir, and another 1 lakh cusecs gets added to it from the catchment of the downstream 1 km length of the channel, then eventually 2 lakh cusecs will be flowing in the channel farther down the 1 km length. This 2 lakh cusecs may prove quite harmful, as it may exceed the safe carrying capacity of the downstream river (below 1 km).

Another possibility for gate operators could then be to release lesser discharge, say 50,000 cusecs, which with 1 km downstream contribution of 1 lakh cusecs, will make up 1,50,000 cusecs, which may not prove dangerous to farther down. But releasing lesser quantity would cause the reservoir water level to rise rapidly, encroaching the 'flood reserve' quickly, finally leading to a stage, wherein you may have to open the entire gates, passing say 4 lakh cusecs, to avoid over-topping and failure of the dam. This 4 lakh cusecs will eventually destroy the entire downstream area, and may prove worser than what would have happened, if 1 lakh cusecs would have been allowed in the beginning itself.

You can, thus, understand and appreciate as to how important it is, to properly man the outflows from a reservoir. The intelligent and timely manipulations, accompanied by proper rain forecasts, may help us to avoid flooding of the downstream area.

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* D.V.C. (Damodar Valley Corporation) was constituted in India in July, 1948, on the line of T.V.A. (Tennesse Valley Authority) of USA. to plan and execute flood control works on river Damodar and its tributaries, which had created havoc in the year 1943, causing serious flood losses of the order of Rs. 8 crores, and cutting off the most important Calcutta city with the rest of the country for about ten weeks, due to breaches in G.T. Road and Rly link.

Consequently, 4 dams were constructed, including the Maithon on river Damodar, Konar on river Konar, and Panchat and Tallaiya on its tributary Barakar, along with a barrage at Durgapur (situated a few km downstream of the confluence of Damodar and Barakar. Konar and *Talliya* are smaller dams located in the upper reaches, whereas, Maithon and Panchat are bigger dams in lower reaches, bearing most of the flood burden.

Maithon is located about 13 km above the confluence, and 21 km northwest of Asansol and 40 km east of Dhanbad. Like Konar, it is partly earthen and partly of concrete. The primary purpose of the dam is flood control; whereas, *irrigation* and *hydropower* are secondary purposes.

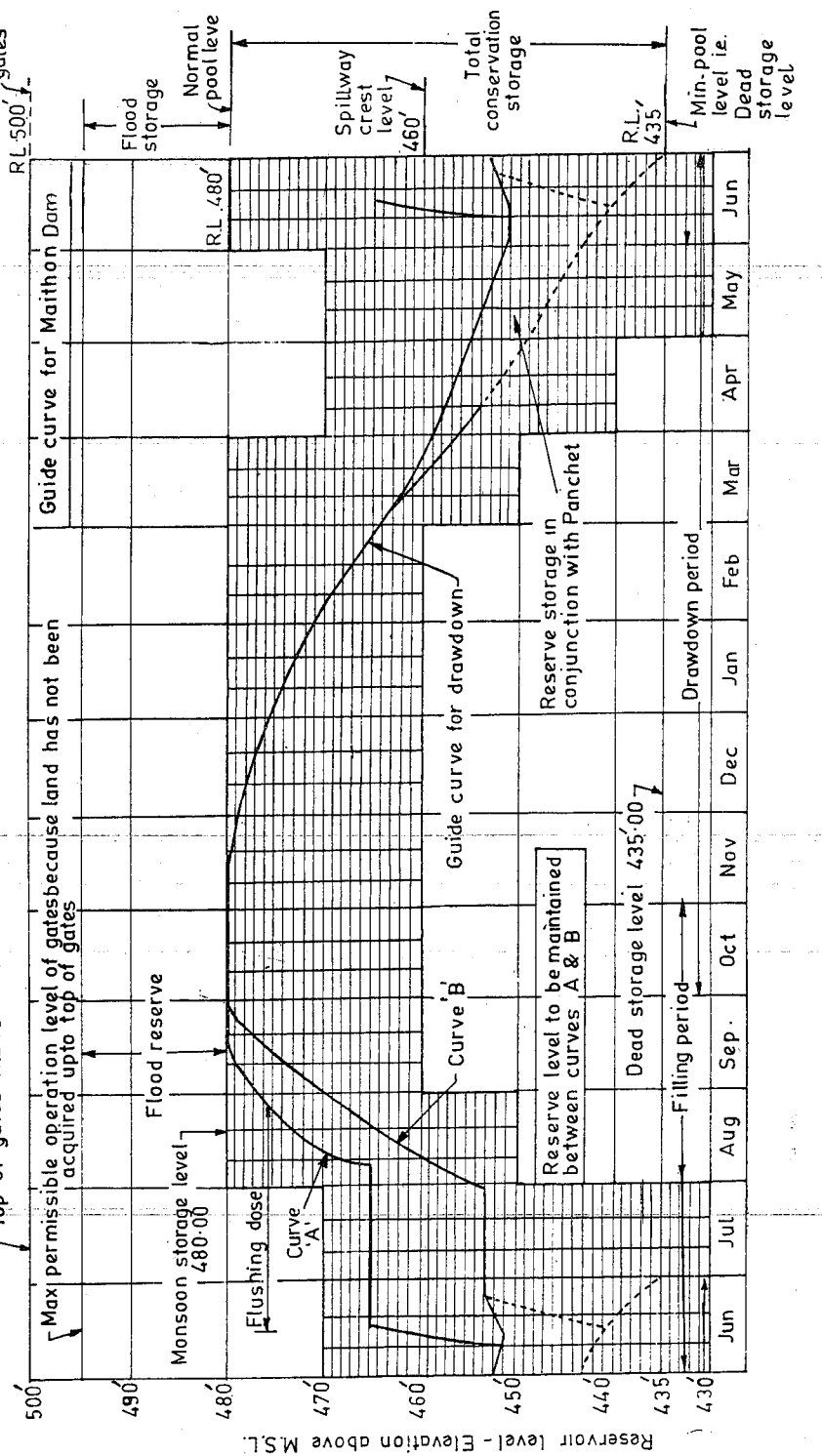


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

18.13. Reservoir Sedimentation

Every river carries certain amount of sediment load. The sediment particles try to settle down to the river bottom due to the gravitational force, but may be kept in suspension due to the upward currents in the turbulent flow which may overcome the gravity force. Due to these reasons, the river carries fine sediment in suspension as suspended load, and larger solids along the river bed as bed load. When the silt laiden water reaches a reservoir in the vicinity of a dam, the velocity and the turbulence are considerably reduced. The bigger suspended particles and most of the bed load, therefore, gets deposited in the head reaches of the reservoir. Fine particles may travel some more distance and may finally deposit farther down in the reservoir, as shown in Fig. 18.24. Some very fine particles may remain in suspension for much longer period, and may finally escape from the dam along with the water discharged through the sluiceways, turbines, spillway, etc.

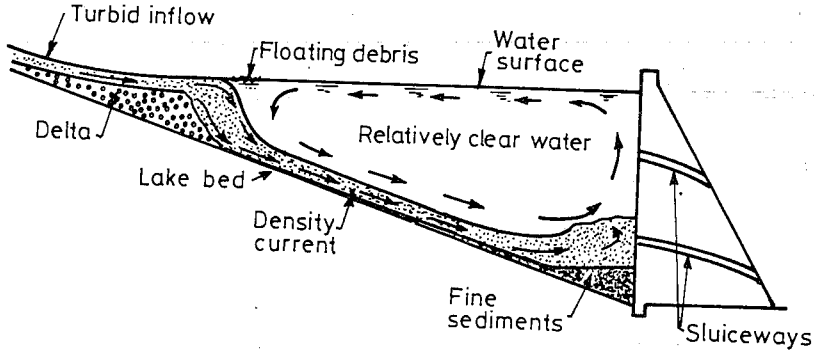


Fig. 18.24. Sediment accumulation in a typical reservoir.

The deposition of sediment in the reservoir is known as 'Reservoir Silting' or 'Reservoir Sedimentation'.

The deposition of the sediment will automatically reduce the water storing capacity of the reservoir, and if this process of deposition continues longer, a stage is likely to reach when the whole reservoir may get silted up and become useless.

Moreover, with the passage of time, the reservoir capacity will go on reducing. Thus, if today at the time of construction, a reservoir can store 10,000 cubic metres of water, tomorrow say after five years, it may be able to store only say 8,000 cubic metres of water. Therefore, in order to see that the capacity does not fall short of requirement ever during the design period, we must take this silting into account. The total volume of silt likely to be deposited during the designed life period of the dam is, therefore, estimated ; and approximately that much of volume is left unused to allow for silting, and is known as **dead storage.** The remainder is known as the *effective storage* or the **live storage.** The dead storage generally varies between 15 to 25% of the total capacity. For example in Bhakra dam, the gross capacity of the dam is 9,344 million cubic metres and the dead storage provided is 2,054 million cubic metres. All the outlets fetching water from the reservoir are provided above the dead storage level.

The importance of this silting can be understood by considering the following example :

Let the total capacity of a reservoir be 30 million cubic metres and the provision of dead storage be 6 million cubic metres. Let the average volume of sediment deposition be 0.15 million cubic metres per year. Then it is evident, that the dead storage will be filled up in $\frac{6}{0.15} = 40$ years, and the total storage in about $\frac{30}{0.15} = 200$ years.

Hence, the usefulness of this reservoir would start reducing after 40 years, and after 200 years it would be nothing but a collection of sand and sediment with no water in it, provided the siltation rate remains constant at 0.15 M.m^3/yr.

18.13.1. Density Currents. In a reservoir, the coarser sediment settles down along the bottom of the reservoir, as the muddy flow approaches the reservoir; while the finer sediment usually remains in suspension, and moves in a separate layer than the clear reservoir water, as shown in Fig. 18.24. This layer of water, containing the fine sediment, moves below the upper clearer reservoir water, as a *density current*, since its density is slightly more than the density of the main body of the reservoir water. *Because of their density difference, the water of the density current does not mix easily with the reservoir water, and maintains its identity for a considerable time.* The density current can thus be removed through the dam sluiceways, if they are located properly and at the levels of the density current. A lot of sediment load can, thus, be passed out of the reservoir, if it is possible to locate the dam outlets and sluiceways in such a fashion, as to vent out the density currents. Trap efficiency of reservoirs may thus be decreased by about 2 to 10%, if it is possible to vent such density currents through the outlets and sluiceways of the dams.

18.13.2. Trap Efficiency. Now, we introduce another very important term called Trap efficiency. Trap efficiency is defined as the percentage of the sediment deposited in the reservoir even inspite of taking precautions and measures to control its deposition.

Therefore, Trap Efficiency (η)

$$= \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}} \qquad ...(18.13)$$

Most of the reservoirs trap 95 to 100% of the sediment load flowing into them. Even if various feasible silt control measures are adopted, it has not been possible to reduce this trap efficiency below 90% or so.

18.13.3. Capacity Inflow Ratio. The ratio of the reservoir capacity to the total inflow of water in it, is known as the capacity-inflow ratio. It is a very important factor, because the trap efficiency (η) has been found to be a function of capacity-inflow ratio *i.e.*

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right) \qquad ...(18.14)$$

The graph obtained for the existing reservoirs between trap efficiency and log of $\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$ has been found to be of the type shown in Fig. 18.25.

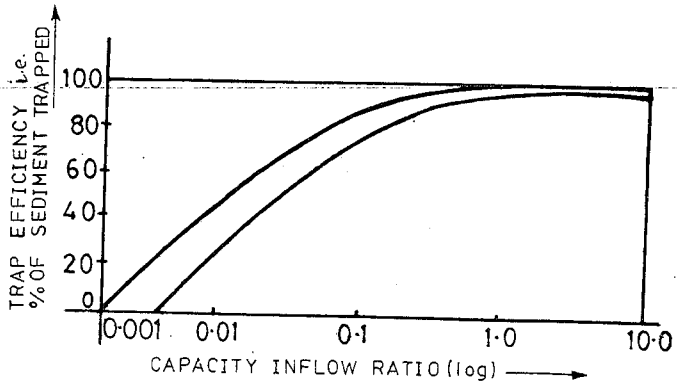


Fig. 18.25.

It is evident from the above curve, that if capacity reduces (with constant inflow), trap efficiency reduces, and hence, lesser sediment is trapped. Therefore, the silting rate in the reservoir shall be more in the beginning, and as its capacity reduces due to silting, the silting rate will reduce. Hence, the complete reservoir-silting may take longer period.

It can also be concluded that for small reservoirs (having small capacity) on large rivers (having large inflow rates), the trap efficiency is extremely low, because the capacity inflow ratio is very small. Such reservoirs silt very little and most of their sediment is passed downstream. On the other hand, large reservoirs on smaller rivers shall silt tremendously and almost complete deposition of sediment may take place.

18.13.4. Silting of Power Reservoirs. In case of reservoirs constructed solely for the purpose of power generation, the silting is comparatively less important. This is because of the fact, that for the proper and efficient functioning of a power reservoir, only a certain minimum head is necessary. This head remains available even after some silt gets deposited. So, only if sufficient water required for power generation remains available, the reservoir's efficiency remains unaffected by silting. But due to silted water, the abrasion of the blades of the turbines may occur very soon, and power production may be stopped over a considerable length of time.

18.13.5. Silting Control in Reservoirs. In order to increase the life of a reservoir, it is necessary to control the deposition of sediment. Various measures are undertaken in order to achieve this aim. The various methods which are adopted can be divided into two parts :

(1) Pre-constructing measures ; and (2) Post-constructing measures.

These measures are discussed below :

(1) **Pre-constructing measure.** They are those measures which are adopted before and during the execution of the project. They are innumerated below :

(*a*) *Selection of Dam Site.* The silting depends upon the amount of erosion from the catchment. If the catchment is less erodible, the silting will be less. Hence, the silting can be reduced by choosing the reservoir site in such a way as to exclude the run off from the easily erodible catchment.

(*b*) *Construction of the Dam in Stages.* The design capacity plays an important role in the silting of a reservoir. When the storage capacity is much less than the average annual runoff entering the reservoir, a large amount of water will get out of the reservoir, thereby, reducing the silting rate compared to what it would have been if the entire water would have been stored. Therefore, the life of a reservoir can be prolonged by constructing the dam in stages. In other words, first of all, the dam should be built lower, and raised subsequently when some of its capacity gets silted up.

(*c*) *Construction of Check Dams.* The sediment inflow can be controlled by building check dams across the river streams contributing major sediment load. These are smaller dams and trap large amounts of coarser sediments. They are quite expensive.

(*d*) *Vegetation Screens.* This is based on the principle that vegetations trap large amounts of sediment. The vegetation growth is, therefore, promoted at the entrance of the reservoir as well as in the catchment. These vegetative covers, through which flood waters have to pass before entering the reservoirs, are known as vegetation screens, and provide a cheap and a good method of silt control.

(*e*) *Construction of Under-sluices in the Dam.* The dam is provided with openings in its base, so as to remove the more silted water on the downstream side.

The sediment concentration will be more at some levels than at others. Therefore, sluices are located at the levels of higher sediment concentration. The method in itself, is not sufficient because the water digs out a channel behind the sluice for movement and leaves most of the sediment undisturbed. Therefore, this is simultaneously supplemented with mechanical loosening and scouring of the neighbouring sediment in order

to increase its effectiveness. But to provide large sluices near the bottom of the dam, is again a structural problem. The use of this method is, therefore, limited.

(2) **Post-constructing Measures.** These measures are undertaken during the operation of the project. They are given below:

(1) *Removal of Post Flood Water.* The sediment content increases just after the floods ; therefore, attempts are generally made not to collect this water. Hence, the efforts should be made to remove the water entering the reservoir at this time.

(2) *Mechanical Stirring of the Sediment.* The deposited sediment is scoured and disturbed by mechanical means, so as to keep it in a moving state, and thus, help in pushing it towards the sluices.

(3) *Erosion Control and Soil Conservation.* This includes all those general methods which are adopted to reduce erosion of soil and to make it more and more stable. This method is the most effective method for controlling siltation, because when the soil erosion is reduced, the sedimentation problem is reduced automatically. But the methods of treating the catchment in order to minimise erosion are very costly. It has been estimated that the investment required for treating 16% of the Indian catchment area is Rs. 1,000 crores. In India, only 1.5% of the catchment area has been treated to minimise silting.

Example 18.16. *The following information is available regarding the relationship between trap efficiency and capacity inflow ratio.*

Capacity inflow ratio	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1.0*
Trap efficiency percent	*87*	*93*	*95*	*95.5*	*96*	*96.5*	*97*	*97*	*97*	*97.5*

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the average annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 2,00,000 tonnes. Assume a specific weight of the sediment equal to 1.2 gm per c.c. The usual life of the reservoir will terminate when 80% of its initial capacity is filled with sediment.

Solution.

Average annual sediment inflow = 2,00,000 tonnes

$$= 2 \times 10^5 \text{ tonnes} = 2 \times 10^{11} \text{ gm}$$

Volume of average annual sediment inflow

$$= \frac{2 \times 10^{11}}{1.2} \text{ c.c.} = \frac{2 \times 10^{11}}{1.2 \times 10^6} \text{ m}^3$$

$$= \frac{0.2}{1.2} \times 10^6 \text{ cubic metres} = \frac{1}{6} \text{ million cubic metre} = \frac{1}{6} \text{ M.m}^3$$

Initial Reservoir Capacity $= 30 \text{ M.m}^3$

Annual flood inflow $= 60 \text{ M.m}^3$

Let us assume that 20% of the capacity, *i.e.* 6 M.m^3 is filled up in the first interval.

Capacity inflow ratio at the start of the interval $= \frac{30}{60} = 0.5$

Trap efficiency at the start of the interval = 0.96.

Capacity inflow ratio at the end of the interval $= \frac{24}{60} = 0.4$

Trap efficiency at the end of interval = 0.955

Average trap efficiency during the interval $= \frac{0.96 + 0.955}{2} = \mathbf{0.9575.}$

Volume of sediment deposited annually till the 20% capacity is filled

$$= \frac{1}{6} \times 0.9575 \text{ M.m}^3$$

$\therefore$ No. of years during which 20% of the capacity, *i.e.* 6 M.m^3 shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9575} \text{ years} = \frac{36}{0.9575} = \mathbf{37.6\ years}$$

Similarly, in the 2nd interval

Capacity inflow ratio at the start $= \frac{24}{60} = 0.4$

Capacity inflow ratio at the end $= \frac{18}{60} = 0.3$

Trap efficiency at the start $= 0.955$
Trap efficiency at the end $= 0.95$
Average trap efficiency $= \mathbf{0.9525.}$

$\therefore$ No. of years during which the next 20% of capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9525} = \frac{36}{0.9525} = \mathbf{37.8\ years.}$$

Similarly, in the 3rd interval

Capacity inflow ratio at start $= \frac{18}{60} = 0.3$

Capacity inflow ratio at the end $= \frac{12}{60} = 0.2$

Trap efficiency at the start $= 0.95$
Trap efficiency at the end $= 0.93$
Average trap efficiency during the interval $= 0.94$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.94} = \frac{36}{0.94} = \mathbf{38.3\ years.}$$

Similarly, in the 4th interval

Capacity inflow ratio at the start $= 12/60 = 0.2$
Capacity inflow ratio at the end $= 6/60 = 0.1$
Trap efficiency at the start $= 0.93$
Trap efficiency at the end $= 0.87$

Average trap efficiency during the interval $= 0.90$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9} = \frac{36}{0.9} = \mathbf{40\ years.}$$

Total probable life till 80% capacity gets filled up

$$= 37.6 + 37.8 + 38.3 + 40.0 = \mathbf{153.7\ years.} \quad \textbf{Ans.}$$

The above calculations of dividing the entire capacity into intervals (20% each in the above case) can also be carried out in a tabular form, as shown below in Table 18.28.

Table 18.28

Capacity		Capacity/inflow $= \frac{\text{Col. (2)}}{60\ \text{Mcum}}$	Trap efficiency η	Av. Trap eff. η_{av} during the interval	Sediment trapped per year; Col. (5) × Av. annual sediment inflow = Col. $5 \times \frac{1}{6}$ M.cum.	Years reqd. to fill up 20% capacity (6 Mcum.) $= \frac{6}{\text{Col. (6)}}$ in years
%	Capacity Vol. in M. cum.					
(1)	(2)	(3)	(4)	(5)	(6)	(7)
100	30	0.5	0.96			
				0.9575	0.1596	36.6
80	24	0.4	0.955			
				0.9525	0.1588	37.8
60	18	0.3	0.95			
				0.94	0.1566	38.3
40	12	0.2	0.93			
				0.90	0.15	40.0
20		0.1	0.8			
						$\Sigma = 153.7$ yrs

Example 18.17. *A proposed reservoir has a capacity of 500 ha-m. The catchment area is 125 km^2, and the annual streamflow averages 12 cm of runoff. If the annual sediment production is 0.03 ha.m/km^2, what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation? The relationship between trap efficiency η (%) and capacity inflow ratio C/I, is as under :*

C/I	0.01	0.02	0.04	0.06	0.08	0.1	0.2	0.3	0.5	0.7
η%	43	60	74	80	84	87	93	95	96	97

(U.P.S.C., Civil Services, 1987)

Solution. Av. annual streamflow = 12 cm of runoff

Area of catchment $= 125\ \text{km}^2 = 125 \times 10^6\ \text{m}^2$

$\therefore$ Annual flood inflow $= (125 \times 10^6) \cdot \frac{12}{100}\ \text{m}^3$

$= 15 \times 10^6\ \text{m}^3 = \mathbf{15\ M.m^3\ (Mcum)}.$

Annual sediment inflow = 0.03 ha-m/km^2 of the catchment

$= 0.03 \times 125$ ha–m $= 0.03 \times 125 \times 10^4\ \text{m}^3 = 3.75 \times 10^4\ \text{m}^3$

$= \frac{3.75}{100} \times 10^6\ \text{m}^3 = \mathbf{0.0375\ M\text{–}m^3\ (Mcum)}.$

It means that 0.0375 Mcum of sediment flows every year into the dam/reservoir site, but the quantum of this, which is trapped in the reservoir, depends upon the average trap efficiency (η) during that year, and this trap efficiency, in turn, depends upon the capacity/inflow ratio.

In the question, the total capacity to be filled up by sediment is 10% of the initial reservoir capacity,

i.e. $10\% \times 5\ \text{Mcum} = 0.5\ \text{Mcum}.$

Now, we have to calculate the time during which this 0.5 Mcum of sediment will get deposited in the reservoir, as follows :

Capacity of the reservoir at the start = 5 Mcum

Capacity of the reservoir at the end

(*i.e.*, when 0.5 Mcum of sediment is filled up) = 4.5 Mcum

$\therefore$ Capacity/inflow at the start $= \dfrac{5\text{ Mcum}}{15\text{ Mcum}} = 0.333$

η at start = 95%. Capacity/inflow at the end $= \dfrac{4.5}{15} = 0.30$

η at the end of the interval = 95%. Average η = 95%

$\therefore$ Sediment load trapped/yr.

$= 0.0375 \times 95\% = 0.035625$

$\therefore$ No. of years during which 0.5 Mcum of sediment will get trapped

$= \dfrac{0.5}{0.035625}$ years = 14.04 years ; say **14 years.**

Hence, after 14 years, 10% reservoir capacity will get filled up. **Ans.**

18.14. Estimating Sediment Load likely to Enter a Proposed Reservoir

The quantum of sediment flowing into a dam reservoir along with river runoff primarily depends upon the erosion characteristics of its catchment and the characteristics of the rainfalls that produce the run off entering the reservoir.

The sediment is basically produced by rains by the process of sheet erosion. The flowing water is the most active agent for erosion of soil from the land. Other agents like *wind, gravity, ice* and *human activities* do help in the erosion process. The rain drops in itself, loosen the soil perticles and break the soil lumps. The action of flowing sheet of water on the land surface helps in eroding the top soil from the ground surface and transport it down to the channels. The erosion caused by rainfall and runoff, thus, constitutes of the following two parts :

(1) **Sheet erosion.** It includes the detachment of geological material from the land surface by the impact of raindrops, and its subsequent removal by overland flow; and

(2) **Channel erosion.** It includes river bank erosion and transportation of the materials by concentrated flow.

Human activities, like *overgrazing of grass land, cutting of forests, forest fires, ploughing of land,* and various *mining & other excavations* etc. have magnified the problem of water erosion in river channels.

18.14.1. Factors Affecting the Erodibility of a Soil. The factors which effect the erodibility of a soil are given below :

(*i*) *Particle size of soil.* Larger the size of soil particles, the lesser would be its chances for erosion.

(*ii*) *Land slope.* The greater is the land slope, the greater is the action of erosive agents, the optimum being at 40° slope.

(*iii*) *Vegetation.* The thicker is the vegetation cover over a soil, the lesser will be the scope of erosion of the soil from the area.

(*iv*) *Presence of Salt and Colloidal Matter in the Soil.* The binding materials like kaolinite, montmorillonite, biotite, etc., do help in increasing the force of cohesion between the soil particles, thereby reducing the erodibility of the soil.

(*v*) *Moisture Content of Soil.* The greater is the moisture content of the soil, the lesser is the scope of its erosion.

(*vi*) *Soil compaction.* The higher is the compaction of soil, the lesser is the chance of its erosion.

(*vii*) *Soil Properties.* The characteristics of the soil, such as soil texture, structure, stratification, permeability, composition etc. do affect the soil binding, which in turn will neutralise the force of weathering agents.

(*viii*) *Human Activities.* Human activities on the land like mining, agricultural operations, construction of projects, land use, etc. do increase the erosion from the given land.

(*ix*) *Rainfall characteristics.* The intensity, duration, quantity and distribution of rainfall over space and time are some of the important factors that affect sediment yield.

18.14.2. Estimation of Sheet Erosion. The sediment yield of a reservoir basically depends on sheet erosion which can be estimated by the following empirical equations :

(1) *Musgrave Equation.* Musgrave (1947) suggested the following equation to compute the annual gross sheet erosion from a catchment, on the basis of 19 widely scattered research stations in USA :

$$E = CR \cdot (S_0/10)^{1.35} (L/72)^{0.35} (P_{30}/1.25)^{1.75} \quad ...(18.15)$$

where E = Erosion of soil lost from the catchment in inches/year

C = the *soil erosion rate,* which varies from 0.43 to 0.53 inch/year depending on the soil type (which depends on texture and permeability of soil)

R = the *cover factor,* which varies from 0.95 for poorly covered land to 0.10 for row crops

S_0 = Land slope in percentage, the default being 10%

L = Length of the land slope in feet

P_{30} = The max. rainfall in inches having 30 min. duration and of 2 year frequency.

It was also stipulated that the erosion value of E computed above, in inch/year, can be multiplied by 150 to obtain the erosion value in ton/year/acre.

(2) *Universal Equation.* Agriculture Research Service of U. S. Deptt. of Agriculture developed a universal equation (1961) to predict erosion from **small catchments.** This universal equation is given as :

$$E_a = R_f \cdot K \cdot (LS)\, C_m \cdot P \quad ...(18.16)$$

where E_a = Average soil loss in ton/acre/year.

(*i*) R_f is **Rainfall-run off factor.** Its value takes into account the effect of rain drop impact as well as the resulting amount and rate of run off. It should also include the cumulative effects of many moderate sized storms as well as the effects of occasional severe storms.

The value of R_f is generally taken as equal to EI (**rainfall erosion index**), which is computed as :

$$E \cdot I = E \times I_{30} \quad ...(18.17)$$

where E = storm energy in 100 ft-ton/acre/inch

I_{30} = Max. 30-minute intensity in inch/hr.

The value of E is further computed as :

$$E = \frac{1}{100}\left[916 + 331 \log I\right] \quad ...(18.18)$$

where E = Storm energy in 100 ft-ton per acre per inch.

I = Storm intensity of the given storm with a limit of 3 inch/hr, since median drop size does not continue to grow beyond this limit.

The *EI* value can, thus, be computed by eqn (18.17) for a given storm. For a specified period, the individual storm *EI* values can be summed up, which provide a numerical measure of the erosive potential of the rainfall within that period (one year). *In this manner, the average annual total of the storm EI values in a particular area is obtained, which is called the rainfall-erosion index for that area,* which equals R_f.

(*ii*) K is **Soil erodibility factor.** Its value is determined experimentally for the given soil type. Representative values for different types of soils have been worked out and listed for different regions of USA by Soil Conservation Service of USA, as shown in Table 18.29.

Table 18.29. Computed K Values for Soils on Erosion Research Stations (After Wischmeier and Smith, 1978).

Soil	*Source of Data*	*Computed K*
Dunkirm silt loam	Geneva, NY	0.69[a]
Keene silt loam	Zanesville, OH	0.48
Shelby loam	Bethany, MO	0.41
Lodi loam	Blacksburg, VA	0.39
Fayette silt loam	LaCrosse, WI	0.38[a]
Cecil sandy clay loam	Watkinsville, GA	0.36
Marshall silt loam	Clarinda, IA	0.33
Ida silt loam	Castana, IA	0.33
Mansic clay loam	Hays, KS	0.32
Hagerstown silty clay loam	State College, PA	0.31[a]
Austin clay	Temple, TX	0.29
Mexico silt loam	McCredie, MO	0.28
Honeoye silt loam	Marcellus, NY	0.28[a]
Cecil sandy loam	Clemson, SC	0.28[a]
Ontario loam	Geneva, NY	0.27[a]
Cecil clay loam	Watkinsville, GA	0.26
Boswell fine sandy loam	Tyler, TX	0.25
Cecil sandy loam	Watkinsville, GA	0.23
Zaneis fine sandy loam	Guthrie, OK	0.22
Tifton loamy sand	Tifton, GA	0.10
Freehold loamy sand	Marlboro, NJ	0.08
Blath flaggy silt loam with surface stones > 2 inches removed	Arnot, NY	0.05[a]
Albia gravelly loam	Beemerville, NJ	0.03

a Evaluated from continuous fallow. All others were computed from rowcrop data.

For soils containing less than 70% of silt and very fine sand, K can be computed as :

$$K = \frac{1}{100}\left[2.1\, M^{1.14}\,(10^{-4})\,(12-a) + 3.25\,(b-2) + 2.5\,(c-3)\right] \quad ...(18.19)$$

where M = particle size parameter, defined as percent silt & very find sand (size 0.1 mm-0.002 mm) times the quantity (100-percent clay)

a = percent organic matter

b = soil texture code used in USDA soil classification

c = profile permeability class

[**Note :** When the silt fraction does not exceed 70%, erodibility varies approximately as the 1.14 power of M, but addition of organic matter content, soil structure, and profile-permeability class as done in Eqn. (18.19) improves the prediction accuracy.]

(*iii*) (LS) = **Topographic Factor.** It is the ratio of soil loss per unit area from a field slope to that from a 72.6 ft length of uniform 9% slope under otherwise identical conditions. For a specified slope and its length, LS can be computed as :

$$(LS) = \left(\frac{\lambda}{72.6}\right)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065) \qquad ...(18.20)$$

where λ = slope length in ft

θ = angle of slope

M = 0.5 if the per cent slope is 5 or more

m = 0.4 on slopes of 3.5 to 4.5%

m = 0.3 for slopes of 1 to 3%

m = 0.2 for uniform slopes of less than 1%.

(*iv*) C_m = **Soil Cover and Management Factor.** It measures the combined effect of all inter-related cover and management variables, including the type of vegetation, plant spacing, the stand, the quality of growth, crop sequence, tillage practices, crop residues, incorporated residues, land use residues, fertility treatment, etc.

Values of C_m for pasture, range, idle land, and woodland for a combination of cover conditions are given in Table 18.30.

Table 18.30. Factor C_m for permanent Pasture, Range, and Idle Land (After Wischmeier and Smith 1978).

Vegetative canopy		*Cover that contracts the soil surface*						
Type and height[b]	*Percent cover*[c]		*Percent ground cover*					
		Type[d]	*0*	*20*	*40*	*50*	*80*	*95+*
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
Tall weeds or short brush with the average drop fall height of 20 in.	25	G	0.36	0.17	0.09	0.038	0.013	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.120	0.003
		W	0.26	0.16	0.11	0.076	0.039	0.001
	75	G	0.17	0.10	0.06	0.032	0.011	0.003
		W	0.17	0.12	0.09	0.068	0.038	0.011
Appreciable brush or bushes, with the average drop fall height of 6 ½ ft	25	G	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.087	0.042	0.011
	50	G	0.34	0.16	0.08	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.082	0.042	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.078	0.040	0.011
Trees, but no appreciable low brush Average drop fall height of 13 ft	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.089	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.087	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.084	0.041	0.011

[a] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

[b] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to the drop fall height and is negligible if fall height exceeds 33 ft.

[c] Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[d] G: cover at the surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in. deep. W: cover at the surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

(v) P = **Support Practice Factor.** Its value depends upon crop land practice such as contour tillage, *strip cropping on the contour*, and *terrace systems*. Values of P have been given for each of these practices by Wischmeier and Smith (19 7 8). In general, tillage and planting on the contours reduce erosion. Table 18.31 gives P values for contouring.

Table 18.31. P values (Eq. 18.16) and Slope-Length Limits for contouring (Wischmeier & Smith)

Land Slope %	*P Value*	*Maximum Length* (feet)*
1 – 2	0.60	400
3 – 5	0.50	300
6 – 8	0.50	200
9 – 12	0.60	120
13 – 16	0.70	80
17 – 20	0.80	60
21 – 25	0.90	50

* Limit may be increased by 25% for residue cover after crop seedlings will regularly exceed 50%.

Since it is difficult to get proper data for the use of Eq. (18.16) for a developing country like India, this equation is not popular in developing countries.

18.14.3. Sediment Measurement by Sample Recorder. The sediment produced by sheet erosion from a catchment may not always reach at the point of measurement; *i.e.* the site of dam reservoir. Some part of sediment may be deposited en-route. *The ratio between the yield of sediment at the measuring site and the gross erosion in the catchment is called the* **sediment delivery ratio.**

Thus, the **sediment yield** is the gross sediment yield minus the quantity of sediment deposited en route. The sediment yield, infact, is important, since it is this sediment which will get deposited in the reservoir, affecting its useful life.

The *sediment yield of a reservoir* can be calculated either by using an appropriate empirical equation, out of the ones developed by various investigators; or by developing a rational appropriate relation between inflow and sediment, on the basis of actual measurements of sediment load at the site of the reservoir. The continuous measurements of suspended load and bed load at the reservoir site for a number of years will help in developing a rational relation (either a mathematical equation or a graphical curve) between sediment and the inflow into the reservoir. We will first of all discuss the method of the actual measurement of sediment and developing an appropriate relation between sediment yield (q_s) and the river discharge (Q) at the given site.

18.14.3.1. Measuring suspended sediment load. Sediment samplers are used to measure suspended sediment at a given site on the river to obtain the most reliable results of sediment yield. The bed load should also be calculated either by using the available empirical equations*, or on an adhoc basic of 2.5-25% of the suspended load, as to calculate the total sediment load (sum of suspended load and bed load).

A typical sediment sampler used in India is shown in Fig. 18.26. A depth integrating hand sampler used for small streams is also shown in Fig. 18.27.

* Please see article 4.9.

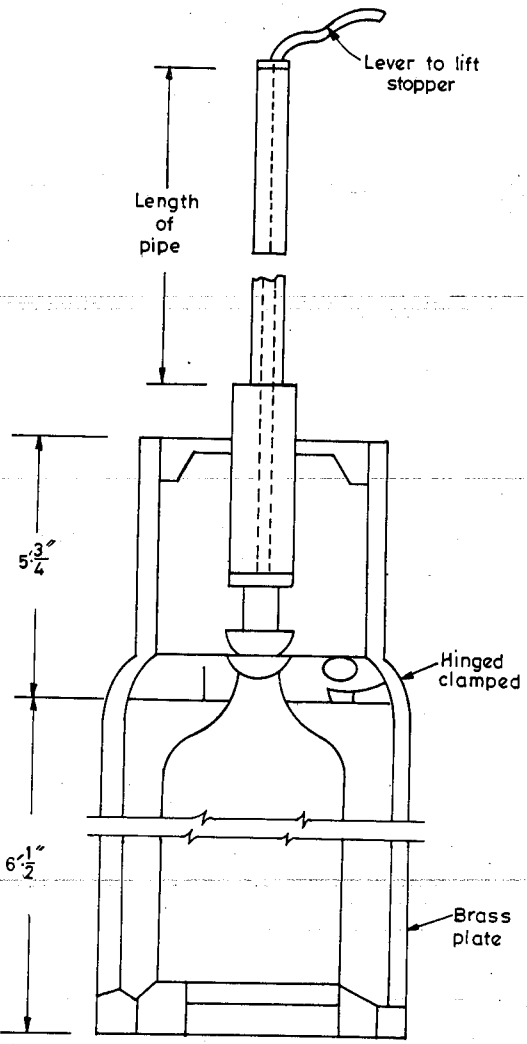


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

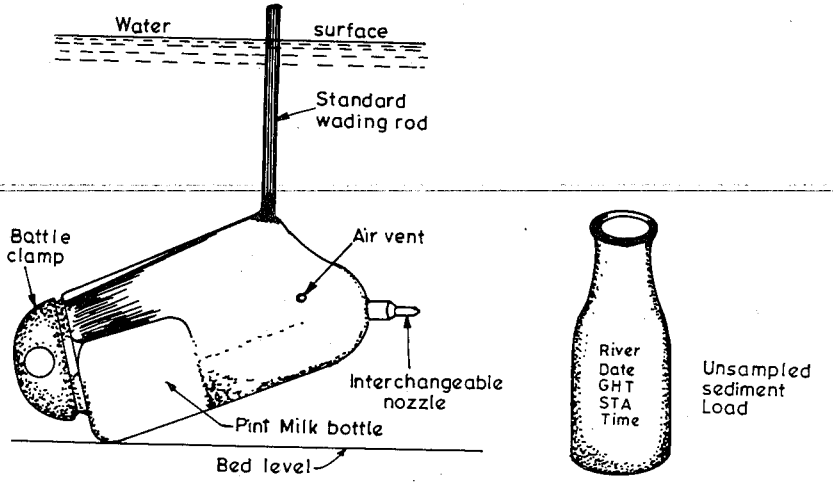


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

The sampler is taken into the stream to a depth of 0.6y below the water surface, or at two depths at 0.2y and 0.8y depths to collect samples of sedimented water in the sampler bottle. The collected sample of sedimented water is then analysed in the laboratory either by a *gravimetric method*, or by *hydrometric method* to determine the quantum of coarse sediment (particle size higher than 0.2 mm), *medium size sediment* (particle size between 0.075 mm to 0.2 mm) and of *fine sediment* (particle size less than 0.075 mm), separately*. Their sum will indicate the total sediment load present in the given volume of water sample. Sediment load present in the water sample is then expressed in ppm (parts for million) as :

$$\text{Sediment load in ppm} = \frac{\text{Dry mass of sediment}}{\text{Total mass of original sample including the mass of sediment \& of water}} \times 10^6 \quad ...(18.21)$$

This value can be converted into t/day by multiplying the average unit wt. of sediment (say 1.2 t/m^3) with the total volume of daily inflow in m^3.

When a large number of such sample records become available for the given site, then a curve can be plotted between the sediment load (q_s) in t/day on x-axis and daily discharge in m^3/s on y-axis, as to obtain a curve known as *sediment rating curve,* as shown in Fig 18.28. Such a plotting is usually done on a log-log paper.

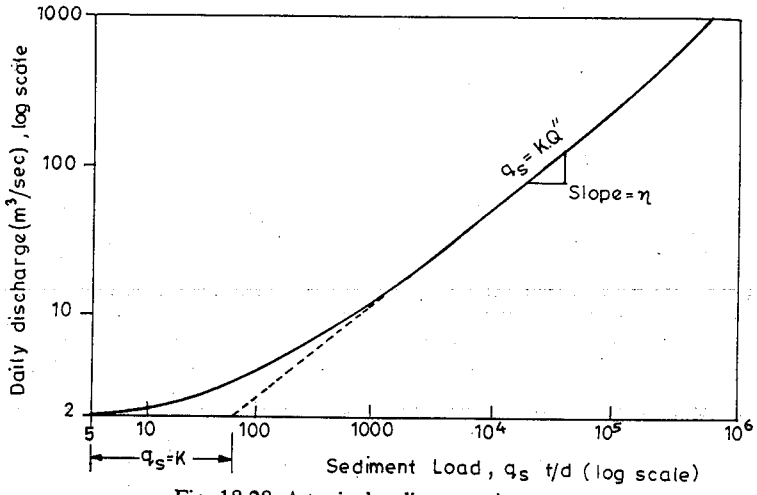


Fig. 18.28. A typical sediment rating curve.

When a standard sediment rating curve is established for a given site, the sediment yield (q_s) can be read out by simply knowing the discharge rate only. However, care must be taken to see that for different seasons of the year, different curves will have to be developed, since the sediment yield of a basin may vary with the season. The curve, for say March (non-monsoon period) may be entirely different from the curve for the month of say August (monsoon season).

* The sample collected by the sampler is first passed through a BSS-100 sieve and the coarse particles retained are taken out and oven dried. Thus, the quantity of coarse sediment (higher than 0.2 mm size) is obtained. Sedimented water passing 200 μ sieve is allowed to stand for 20 minutes, so that the finer particles settle down. The settled mass is removed by the process of decantation (pouring out water from the settled tank). The settled residue is dried and weighed, as to get the mass of fine sediment (0.075 mm – 0.2 mm). Poured water contains sediment of still finer particles. To isolate this, the sample is filtered through a filter paper, and the quantity retained therein is dried and weighed. This gives the mass of fine sediment (particle size < 0.075 mm). The summation of all the three masses will give the total mass of sediment.

The sediment rating curves can be used to compute daily sediment load for the given daily discharge values, and their summation can give us the monthly or the annual sediment load.

18.14.3.2. Mathematical equation for a sediment rating curve. The sediment rating curve (straight line portion) giving sediment load in tonnes/day (q_s) w.r. to daily discharge (Q) in m^3/sec can be expressed by a mathematical equation of the form :

$$q_s = K \cdot Q^n \qquad ...(18.22)$$

Taking log on both sides,

$$\log q_s = \log K + n \cdot \log Q$$

This equation is similar to the form

$$y = a + bx,$$

where $\log q_s$ is plotted on y-axis and $\log Q$ on x-axis.

$$n = b \qquad ...(18.23)$$

$$\log K = a \qquad ...(18.24)$$

A mathematical solution for such an equation can be obtained by the statistical **method of least squares,** where in the various known values of x (*i.e.* $\log Q$) and y (*i.e.* $\log q_s$) are analysed to estimate a and b values as :

$$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.25)$$

$$b = \frac{N \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.26)$$

By computing values of a and b, values of n and K become known to finally compute the relation between q_s and Q by Eq. (18.22).

The use of this method will become clear when we solve example 18.18.

Example 18.18. *A reservoir has the following sediment and discharge data :*

Year	*81*	*82*	*83*	*84*	*85*	*86*	*87*	*88*
Discharge ($M\,m^3$)	*1430*	*3850*	*2050*	*6510*	*2880*	*1120*	*6050*	*2220*
Sediment load (M.t) as measured by silt sampler	*2.65*	*5.82*	*3.60*	*7.15*	*5.22*	*1.95*	*6.88*	*3.94*

Calculate the average total sediment load/year/100 sq. km of the catchment at the site. Develop a regression relation and predict the total and observed sediment yield for the inflow of 3450 $M.m^3$ for the year 1978. Take the catchment area at the site as 3050 sq. km. What is the total sediment yield for 100 years ? Assume bed load as 10% of suspended load.

Solution. Average sediment load measured by silt sampler

$$= \frac{2.65 + 5.82 + 3.6 + 7.15 + 5.22 + 1.95 + 6.88 + 3.94}{8}$$

$$= \frac{37.21}{8} \text{ Mt/year} = 4.651 \text{ Mt/yr.}$$

Since sediment load is measured by a silt sampler, the given values of sediment are of suspended load.

Now, assume Bed load = 10% of suspended load

$\therefore$ Total load = Suspended load + Bed load

$\therefore$ Total sediment load = $1.1 \times 4.651 = 5.116$

or $(q_s) = 5.116$ Mt/yr.

Sediment deposited in a reservoir consolidates gradually due to the increasing silt load on it every year and the wt. of water above it. Assuming the average sediment unit weight (sediment in a reservoir consists of sand, silt and clay in water) as 1.2 t/m^3, we have

$\therefore$ Total silt load/year $= 5.116$ Mt

$$q_s = \frac{5.116}{1.2} \text{ M·m}^3 = \mathbf{4.263 \text{ M m}^3}$$

Catchment area $= 3050$ sq. km

$\therefore$ Sediment load/100 sq. km/year $= \dfrac{4.263 \text{ M m}^3}{3050} \times 100$

$= \mathbf{0.1397 \text{ M m}^3/100 \text{ sq. km/year}}$ **Ans.**

Total sediment yield in 100 years = 4.263 M m^3/yr. × 100 yr. = **426.3 M m^3** **Ans.**

In order to develop a regression type non-linear equation between total sediment load (q_s) and discharge (Q), represented as $q_s = K \cdot Q^n$, we carry out the required calculations in table 18.32 to evaluate the values of a and b by Eqn. (18.25) and (18.26), as :

Table 18.32. Computations to develop Equation $y = a + bx$ by the method of least squares.

year	*Discharge* Q ($M \cdot m^3$)	*Suspended Sediment measured in M.t*	*Total sediment i.e.* q_s *in M m*^{3} $col\,(3) \times \frac{1.1^*}{1.2}$	$\log Q = x$ $\log col\,(2)$	x^2 $(col\,5)^2$	$\log q_s = y$ $\log col\,(4)$	$x \cdot y$ $col\,(5) \times col\,(7)$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
81	1436	2.65	2.429	3.1553	9.9561	0.3855	1.2164
82	3850	5.82	5.335	3.5855	12.8555	0.7271	2.6070
83	2050	3.60	3.300	3.3118	10.9677	0.5185	1.7172
84	6510	7.15	6.554	3.8136	14.5433	0.8165	3.1138
85	2880	5.22	4.785	3.4594	11.9674	0.6799	2.3520
86	1120	1.95	1.788	3.0492	9.2977	0.2522	0.7690
87	6050	6.88	6.307	3.7818	14.3017	0.7998	3.0247
88	2220	3.94	3.612	3.3464	11.1981	0.5577	1.8663
			Σ	27.503	95.0875	4.7372	16.6664

* q_s (Total sediment in Mm^3) $= \dfrac{q_s \text{ in M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)$** $= \left(col\,(3) \times \dfrac{1.1}{1.2} \right)$

** Total sediment = 1.1 (Observed sediment by soil sampler)

Using $$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N\Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.25)

we get $$a = \frac{4.7372 \times 95.0875 - 27.503 \times 16.6664}{8 \times 95.0875 - (27.503)^2}$$

or $$a = \frac{-7.9275}{4.2850} = -1.8500$$

But $\log K = a = -1.8500 \quad \therefore \quad K = 0.014$

Also $$b = \frac{N \cdot \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.26)

$$= \frac{8 \times 16.6664 - 27.503 \times 4.7372}{8 \times 95.0875 - (27.503)^2} = \frac{3.0440}{4.2850} = 0.7104$$

But $b = n = 0.7104$

Regression equation is hence given as :

$$q_s = K \cdot Q^n = 0.014\,(Q)^{0.7104}$$ **Ans.**

Total sediment yield for the year 1978 having inflow of 3450 M m^3

$$= 0.014\,(3450)^{0.7104} = \mathbf{4.565\ M\ m^3}$$

$$= 4.565\ \text{M m}^3 \times 1.2\ \text{t/m}^3 = \mathbf{5.48\ Mt}$$ **Ans.**

Observed or suspended sediment load for the year 1978

$$= \frac{5.48}{1.1}\ \text{Mt} = \mathbf{4.98\ Mt}$$ **Ans.**

18.14.3.3. Empirical equations for total sediment yield. The following empirical equations have been developed by several investigators, for estimating the annual sediment yield of a reservoir.

(1) *Swami's Regression Equation.* Swamy and Garde (1977) have proposed a relation correlating the cumulative volume of sediment deposited in a reservoir with the cumulative volume of water inflow, and initial bed slope of the river, as

$$V_s = C \cdot B \cdot (V_{ci})^{0.94}\,(S_0)^{0.84}$$ ...(18.27)

where V_s = cumulative vol. of sediment deposited in the reservoir in M · m^3

C = Regression constant with safe value of the order of 1.16. However a value less than 1.16 may be adopted depending on the reservoir

B = Width of the reservoir at full reservoir level in m

V_{ci} = Cumulative vol. of inflow per unit width B of the reservoir

S_o = Bed slope of the river.

(2) *Jogelkar's Equation.* An equation proposed by Jogelkar (1960) is given as :

$$Q_s = 0.59\,(A)^{-0.24}$$ ...(18.28)

where Q_s = Annual silting rate in M · m^3 per 100 sq. km of catchment area

A = Catchment area in sq km

(3) *Khosla's Equation.* Khosla (1953) has proposed the following empirical equation.

$$Q_s = 0.323\,(A)^{-0.28} \qquad ...(18.29)$$

where Q_s = Annual siltation rate in $M \cdot m^3/100$ s.q km/year

A = Catchment area in sq. km.

This equation always gives lower estimate of sediment yield at a site.

(4) *Varshneys Equations.* Varshney and Raichur have proposed the following equations for calculating sediment yield for an ungauged basin.

(*i*) *Up to 130 sq. km catchment for mountainous rivers*

$$Q_s = 0.395\,(A)^{-0.311} \qquad ...(18.30)$$

where Q_s = Annual sediment yield rate in $M \cdot m^3$ per 100 sq. km of catchment

A = catchment area in km^2.

(*ii*) *Rivers draining plain area up to 130 sq. km*

$$Q_s = 0.392\,(A)^{-0.302} \qquad ...(18.31)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*iii*) *For area greater than 130 sq. km for North Indian catchment*

$$Q_s = 1.534\,(A)^{-0.264} \qquad ...(18.32)$$

Q_s & A have the same meaning as in Eq. (18.30).

(*iv*) *For South Indian Rivers up to 130 sq. km*

$$Q_s = 0.46\,(A)^{-0.468} \qquad ...(18.33)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*v*) *For areas greater than 130 sq. km for South Indian catchments*

$$Q_s = 0.277\,(A)^{-0.194} \qquad ...(18.34)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(5) *Using Known Data of Similar Catchment.* Sediment yield of an unmeasured watershed Q_{s_2} can be computed from sediment yield of measured catchment Q_{s_1} of similar topography, land cover and land use, on area proportion basis, as

$$Qs_2 = Qs_1 \left(\frac{A_2}{A_1}\right) \qquad ...(18.35)$$

Example 18.19. *Estimate the sediment load in tonne at the proposed dam site in North India with the following data using various empirical equations :*

Catchment area = 1830 sq. km

Width of reservoir at FRL = 560.0 m

River slope at the dam site = 0.006

Average inflows at the site are as follows :

Year	*1982*	*1983*	*1984*	*1985*	*1986*	*1987*	*1988*	*1989*	*1990*
Inflow $M \cdot m^3$	*2210*	*1290*	*1640*	*1780*	*2150*	*1980*	*2540*	*1285*	*1620*

Assume annual siltation rate per 100 sq. km from a similar catchment of 3050 sq. km to be 10.35 M · m³/100 sq. km.

Solution. The sedimentation rate is worked out by using various equations as described in the previous article.

(1) *By Swamy's Regression Method :* The computation is carried out in Table 18.28 by using Eqn. (18.27) as :

$$V_s = C \cdot B\,(V_{ci})^{0.94}\,(S_0)^{0.84}$$

where $C = 1.16$

$B = 560$ m

$S_0 = 0.006$

$\therefore$ V_s = cumulative vol. of sediment deposited in M · m³

$$= 1.16 \times 560\,(V_{ci})^{0.94} \times (0.006)^{0.84}$$

$$= 8.837\,(V_{ci})^{0.94}$$

where V_{ci} is the cumulative vol. of annual inflows per unit width B of reservoir, over the given years.

The computations are carried out in Table 18.33, which is self explanatory.

Table 18.33. Computation of sediment Load by Swamy's Regression Method

S. No. (1)	*Year* (2)	*Inflow in M · m³* (3)	*Cumulative inflow in M m³* (4)	V_{ci} = *cumulative inflow per unit width B of reservoir* = $\frac{col\,(4)}{560}$ (5)	$V_s = 8.837\,(V_{ci})^{0.94}$ $= 8.837\,(col\,5)^{0.94}$ M · m³ (6)
1	1982	2210	2210	3.946	32.11
2	1983	1290	3500	6.250	49.48
3	1984	1640	5140	9.179	71.01
4	1985	1780	6920	12.357	93.91
5	1986	2150	9070	16.196	121.10
6	1987	1980	11050	19.732	145.80
7	1988	2540	13590	24.268	177.11
8	1989	1285	14875	26.563	192.81
9	1990	1620	16495	29.455	**212.48**

Q_s = sedimentation rate per 100 sq. km

$$= \frac{V_s \text{ (i.e. cumulative vol. of silt)}}{\text{No. of years (i.e. 9)}} \times \frac{100 \text{ sq. km}}{C \cdot A \text{ of } 1830 \text{ km}^2}$$

$$= \frac{212.48}{9} \times \frac{100}{1830}$$

or $Q_s = \mathbf{1.29\ M\ m^3/100\ sq.\ km}$ **Ans.**

(2) *By Jogelkar's Equation*

$$Q_s = 0.59\ (A)^{-0.24}$$

$$= 0.59\ (1830)^{-0.24}$$

$$= \mathbf{0.10\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(3) *By Khosla's Equation*

$$Q_s = 0.323\ (A)^{-0.28}$$

$$= 0.323\ (1830)^{-0.28}$$

$$= \mathbf{0.04\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(4) *By Varshneys Equation for Norths Indian catchment exceeding 130 sq. km.*

$$Q_s = 1.534\ (A)^{-0.264}$$

$$= 1.534\ (1830)^{-0.264}$$

$$= \mathbf{0.211\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(5) *From similar catchment.*

$Q_{s_1} = 4.56$ M m^3/100 sq. km

$A_1 = 3050$ sq. km

$A_2 = 1830$ sq. km

$$Q_{s_2} = Q_{s_1} \times \left(\frac{A_2}{A_1}\right)$$

$$= 0.35 \times \left(\frac{1830}{3050}\right) \text{M m}^3/100 \text{ sq. km}$$

$$= \mathbf{0.21\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

18.15. Reservoir Sedimentation Studies on Existing Reservoirs

Sedimentation of storage reservoirs is a natural process, since large part of the silt eroded from the catchment and transported by the river, gets deposited on the bed of the reservoir. This causes reduction in the live as well as dead storage capacities of the reservoir. Progressive loss of capacity due to sediment accumulation results in reduced benefits and may even cause operational problems. *It, therefore, becomes necessary to monitor the sedimentation rates in the existing reservoirs at regular intervals, to help in planning and executing suitable remedial measures for controlling sedimentation in order to prolong the life of the reservoir and its benefits.*

Regular monitoring and updating of elevation-capacity curve of the reservoir immensely helps in better water management. With this aim, *conventional hydrographic surveys* are conducted at regular intervals at the existing reservoirs to determine the available capacities at different elevations, and to help compute the sedimentation volume at such regular intervals. Such conventional surveys will require computation

of water spread area at different water levels, and is quite a tedious and a costly process. **Remote sensing techniques** do offer a modern answer to the costly conventional surveys, as it offers a great potential for application in capacity evaluation of medium to large reservoirs. From the data provided by the remote sensing satellites, it has now been possible to compute loss of reservoir capacity due to sedimentation, and its distribution. The results obtained from this technique have been found to be quite comparable with those obtained from the costly and cumbersome conventional methods. One of the greatest advantage of this technique is that the capacity evaluation could be easily computed on yearly basis.

The methodology involved in this technique requires the use and analysis of the *satellite imageries* provided by the Remote Sensing satellites, which collect the data of the Earth surface features in different bands at regular intervals. In the case of Indian Remote Sensing Satellites IRS-1A and IRS-1B* (both identical satellites), this periodicity is 22 days for Indian sub-continent. These two satellites together are thus capable to provide us data of all our reservoirs at 11 days interval. The third satellite of this series IRS-1C has also been launched recently on 28.01.1996 and helps in providing better pictures even of cloud bound areas. IRS-1D has further been launched on 29.09.1997, through our first Indian rocket launcher.

Due to the water withdrawals from an existing reservoir, its water spread area goes on changing throughout the year. The reservoirs are generally full just after the monsoon period in October, and get depleted to almost dead storage/minimum drawdown level just before start of monsoon season in May or June, every year. The satellite data of various dates during the period from October to May, provide us an array of water spread areas between maximum water level (i.e. around the FRL) and the minimum reservoir level (i.e. around the dead storage level/minimum drawdown level). From the whole set of the satellite data, a few of them which are cloud free and of good quality and representative of the whole range of reservoir levels at close intervals, are selected for analysis.

The method of analysis depends upon the data products. The selected CCT's/FCC's of various dates are analysed for determining the water spread areas. The corresponding water levels are obtained from the daily gauge record of the reservoir. From these, the water volumes between two consecutive water levels are computed using Prismoidal or any appropriate formula. Volume of water below the minimum water level (as recorded by the satellite) and the "new zero"** elevation, are estimated based upon the previous hydrographic surveys. In case, these informations are not available in the hydrographic survey data, then the elevation-area relationship obtained from the hydrographic surveys as well as the one obtained from the satellite data interpretation, should be extended to get the new zero elevation, and then the volume between the minimum mapped water level and this new zero level is estimated. After this, the cumulative water volume at each reservoir level is computed, and then the revised elevation-area-capacity curve is drawn. By comparing the original area-capacity curve or any other such curve, the total sediment volume and its distribution can be computed.

18.16. Observed Sedimentation Rates for Various Important Indian Reservoirs

The observed sedimentation rates for various important dam reservoirs in India are indicated in col. (6) & (7) of Table 18.34.

* IRS-1A was launched on 19.03.1988, and IRS-1B was launched on 23.08.1991.

** The minimum reservoir bed level is raised due to sedimentation, which is termed as the new zero.

Table 18.34. Annual Sedimentation Rates of Various Indian Reservoirs

S. No. (1)	*Name and Location of reservoir* (2)	*Catchment area in 100 sq. km* (3)	*Capacity of reservoir at FRL in* $M \cdot m^3$ (4)	*Surface area at MRL* $M \cdot m^2$ (5)	*Annual sediment rate in ham/100 sq. m* (6)	*Annual volume of sediment deposit* $M \cdot m^3$ $\frac{col\ (6)}{100} \times col\ (3)$ (7)	*Dead storage capacity provided in* $M \cdot m^3$ (8)
1	Bhakra *i.e.* Govind Sagar) (HP)	568	9351	169	6.00	**34.08**	95
2	Gandhi Sagar (MP)	226	7413	660	10.08	22.71	586
3	Hirakund (Orissa)	834	8146	725	3.89	32.44	2318
4	Lower Bhawani (TN)	61.5	929	—	4.10	2.52	0
5	Maithan (DVC)	63	1275	—	13.02	8.20	165
6	Matatila (UP)	207.5	883	—	3.50	7.26	50
7	Mayurkshi (WB)	18.6	617	—	**20.09**	3.74	68
8	Nizam Sagar (AP)	216.94	715	130	6.57	14.25	—
8	Panchat (DVC)	111	1475	153	9.92	11.01	170
10	Ramganga (UP)	30.76	2448	—	17.30	5.32	395
11	Sriram Sagar or Shivaji Sagar (AP)	8.19	3454	—	15.20	1.24	849
12	Tawa (MP)	—	2312	—	8.10	—	263

18.17. Reservoir Losses

Huge quantity of water is generally lost from an impounding reservoir due to *evaporation, absorption,* and *percolation.* Depending upon which, the following losses may occur from such a reservoir :

1. *Evaporation losses ;*
2. *Absorption losses ;* and
3. *Percolation losses* or *Reservoir leakage.*

These losses are discussed below.

18.17.1. Evaporation Losses. The evaporation losses from a reservoir depend upon several factors, such as : *water surface area, water depth, humidity, wind velocity, temperature, atmospheric pressure* and *quality of water* as discussed in article 7.34.2.3. The evaporation loss from a reservoir under the given atmospheric conditions can be easily estimated by measuring the standard pan evaporation and multiplying the same by the pan coefficient. The pan coefficients and various types of pans in use are given in article 7.34.3.4.

The evaporation losses become very significant in a hot and humid country like India ; and realistic estimation of these losses is quite important. These losses in fact vary from place to place and from season to season, and hence monthly values of these losses are usually determined. Typical average values of these losses for North and South India are given in table 18.35.

Table 18.35. Monthly Reservoir Evaporation Losses

Month	*Losses in cm*	
	North India	*South and Central India*
January	7	10
February	9	10
March	13	18
April	16	23
May	27	25
June	24	18
July	18	15
August	14	15
September	14	15
October	13	13
November	9	10
December	8	10
Total	172	182

On the basis of a review conducted on 130 sample reservoirs, the Central Water Commission, in 1990, has, however, estimated the *average annual evaporation loss to be 225 cm ;* and the total water lost from all the existing reservoirs to be 27000 Mm^3 per annum. What a tremendous waste of precious water!

In order to control such large scale wastage of water, several methods have been devised by engineers and scientists. All these methods are based upon the efforts made to reduce the evaporation rate from the surface of the water bodies by physical or chemical means, since the basic meteorological factors affecting evaporation cannot be controlled under normal conditions. *The following methods are generally used for evaporation control :*

1. Wind breakers
2. Covering of the water surface
3. Reduction of the exposed water surface
4. Use of underground storage rather than the use of surface storage
5. Integrated operation of reservoirs
6. Use of chemicals for retarding the evaporation rate from the reservoir surface.

Out of all these methods, the last method has evoked the maximum response from all over the world, and has been considered to be the only practical solution for conservation of fresh water, inspite of its various limitations and disadvantages in high cost of application in normal conditions. The use of chemicals, called **Water Evapo-Retardants (WERs)**, for controlling the evaporation rate from the surface of reservoirs is therefore, discussed here in details.

A non toxic chemical, capable of forming a thin *monomolecular film* over the water surface, is generally spread over the reservoir water surface in powder, liquid or emulsion form. *The resulting film prevents energy inputs from the atmosphere, thus reducing evaporation. Such a film, however, allows the passage of enough air through it, to avoid any harmful effects on the aquatic life due to shortage of oxygen.*

Fatty alcohols of different grades like : Cetyl alcohol ($C_{16}.H_{33}.OH$) popularly called *hexa decanol*, Stearyl alcohol ($C_{18}.H_{37}.OH$) popularly called *Octadecanol*, and Behenyl alcohol ($C_{22}.H_{45}.OH$) called *docosanol*, or a mixture of these chemicals, have been generally used and found to be quite suitable. These chemicals should, however, be 99% pure for getting the desired properties of *monolayer*. National Chemical Laboratory, Pune, has developed one more compound by synthesising alkoxy ethanols.

In general, all such chemical compounds should possess the *following properties* :

(*i*) the chemical compound (WER) should be tasteless, odourless, non-toxic, non-inflammable, and should not produce any effect on the quality of water.

(*ii*) the chemical should easily spread and form an even compact cohesive and efficient monomolecular film on the water surface.

(*iii*) the thin film formed by the chemical should be pervious to oxygen and carbon dioxide, but tight enough to prevent escape of water molecules.

(*iv*) the thin film formed by the chemical should be durable, and should be able to re-seal itself, when broken due to external disturbances such as wind, waves, etc.

(*v*) the chemical and the film formed by it should not be adversely affected by the water borne bacteria, proteins and other impurities present in the water body.

The use of chemical WERs has, however not been found to be cost effective for mass scale use, and has further not been found to be suitable under the following conditions :

(*a*) when the wind velocities exceed 10 km/hr or so.

(*b*) when the temperature rises above 40°C or so.

(*c*) when the size of the water body is relatively large.

Development of cheaper WERs capable of withstanding higher wind speeds upto about 20 km/hr and having strong cohesive forces and properties of self spreading and re-uniting to maintain the monolayer in resilient state even at high wind velocities, is therefore of vital importance. Moreover, the life of the film formed, must be longer, so as to reduce the frequency of application to about 3 to 7 days from its present frequency of 24 hours. Development of such chemical WERs is the subject matter of present research.

Other long term evaporation control measures like *plantation of trees to act as wind breakers**, *reduction of exposed water surface by covers, underground storage of water, integrated operation of reservoirs,* etc. have been employed in some parts of the country. The effectiveness and economics of these methods are, however, yet to be established.

In India, the water conservation methods are presently being adopted only in draught prone and scarcity areas, since large scale use of such methods on all the reservoirs of the country is not found to be economical or practically unfeasible due to their large size and adverse meteorological factors.

18.17.2. Absorption Losses. These losses do not play any significant role in planning, since their amount, though sometimes large in the beginning, falls considerably as the pores get saturated. They certainly depend upon the type of soil forming the reservoir.

18.17.3. Percolation Losses or Reservoir Leakage. For most of the reservoirs, the banks are permeable but the permeability is so low that the leakage is of no

* Trees having lesser evapotranspiration should only be chosen and identified.

importance. But in certain particular cases, when the walls of the reservoir are made of badly fractured rocks or having continuous seams of porous strata, serious leakage may occur. Sometimes, pressure grouting may have to be used to seal the fractured rocks. The cost of grouting has to be accounted in the economic studies of the project, if the leakage is large.

18.18. Reservoir Clearance

The removal of trees, bushes and other vegetation from the reservoir area is known as reservoir clearance. It is an expensive operation and difficult to be justified on cost-benefit considerations. Non-clearance of such vegetation may lead to the following troubles :

(*i*) decay of organic material may create undesirable odours and tastes, and hence, becomes important for water supply reservoirs.

(*ii*) trees projecting above the water surface may create undesirable appearance, when the reservoir is to be used for recreation and tourism purposes.

(*iii*) bushes, trees, etc. will float and may create debris problems at the dam.

18.19. Selection of a Suitable Site for a Reservoir

It is almost impossible to select a perfect ideal reservoir site. But its selection is guided by the following factors :

(*i*) A suitable dam site is available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(*ii*) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(*iii*) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(*iv*) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility of weed growth.

(*v*) The reservoir site must have adequate capacity.

(*vi*) Too much silt laiden tributaries should be avoided as far as possible.

(*vii*) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

18.20. Reservoir Induced Seismicity

During the recent times, a very strange phenomenon has been observed in several dams of the world. What actually has happened is that the reservoir basins or areas which were seismologically inactive, started showing seismic activities, as the reservoir was filled up with water. The magnitude of these earthquakes is found increasing with the filling of the reservoir, giving severe shocks when the reservoir is full. In all such cases, the epicentre is always seen to be located within the reservoir or along its border.

An important Indian example of such an earthquake has been in the case of Koyna dam (Maharashtra), which is situated in the seismologically inactive zone in Peninsular India. However, when the dam got ready and water started collecting (1962) in the reservoir, the earthquakes also started occurring (1963). The frequency and intensity of these earthquakes had gone on increasing, as more and more water went on collecting in the reservoir. On 10th December, 1967, the severest shock occurred with a magnitude of about 6.5, followed by three more shocks of decreasing magnitudes. The epicentres of all these shocks were traced to be within the reservoir area.

Some other examples of such earthquake sites, noticed outside India, are at *Lake Mead* (U.S.A.) ; *Grand Val Lake* (France) ; *Vorgorno Lake* (Switzerland) ; etc.

This phenomenon has been studied by various seismologists, and it has been suggested by some that in most such cases, *there must have existed some inactive faults in the reservoir basin area, which became active again due to the extra-ordinary load of the reservoir water,* thus causing displacements along these faults and consequently resulting in earthquakes. Some other seismologists have suggested that these earthquakes are caused due to *increased pore pressure* in the adjoining rocks, which lowers their shearing strength, resulting in the release of tectonic strain.

Based on these explanations, the various preventive methods suggested for preventing or reducing such earthquakes, include :

(*i*) *filling of the reservoir to a limited safe level ;*

(*ii*) *reducing pore pressure by draining out water from weaker adjoining rocks ; and*

(*iii*) *to actively explore the dam site for the absence of inactive faults before selecting the same.*

Such earthquakes, however, show a decreasing tendency with time. This phenomenon is indeed very complex and interesting, and still needs further research.

18.21. Economic Height of a Dam

The economic height of a dam is that height of the dam, corresponding to which, the cost of the dam per unit of storage is minimum. For this purpose, the estimates are prepared for construction costs, for several heights of the dam, somewhat above and below the level at which the elevation-storage curve shows a fairly high rate of increase of storage per unit rise of elevation, keeping the length of the dam moderate. The construction cost is found to increase with the dam height, as shown in Fig. 18.29.

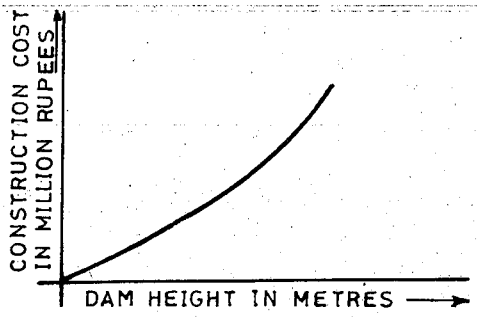


Fig. 18.29

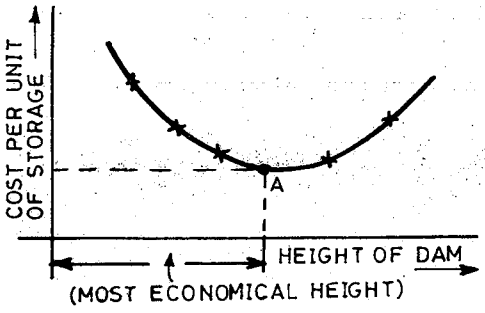


Fig. 18.30

For each dam height, the reservoir storage is known from the reservoir-capacity curve. The construction cost per unit of storage for all the possible dam heights can then be worked out and plotted, as shown in Fig. 18.30.

The lowest point A on this curve, gives the dam height for which the cost per unit of storage is minimum, and hence, most economical.

Example 18.20. *The construction costs for certain possible heights of a dam at a given site have been estimated and are tabulated in the table below. The storage capacity for all these dam heights are also given.*

S.No.	Height of the dam in metres	Construction cost in million Rs.	Storage in million cubic metres
(1)	(2)	(3)	(4)
1	10	4	50
2	20	8	110
3	30	12	180
4	40	18	250
5	50	27	350
6	60	39	500
7	65	50	600

Determine the most economical height of the dam from purely construction point of view.

Solution. The given table is extended, so as to workout the cost per million cubic metre of storage, as shown in col. (5) of Table 18.36.

Table 18.36

S.No.	Height of the dam in metres	Construction cost in M.Rs.	Storage in $M.m^3$	Cost per unit of storage $= \frac{Col.\ 3}{Col.\ 4}$
(1)	(2)	(3)	(4)	(5)
1	10	4	50	0.080
2	20	8	110	0.073
3	30	12	180	0.067
4	40	18	250	0.072
5	50	27	350	0.077
6	60	39	500	0.078
7	65	50	600	0.083

The cost per unit of storage is plotted against the height of the dam, as shown in Fig. 18.31. The most economical height is the lowest point of this curve, and it works out to be **30 metres. Ans.**

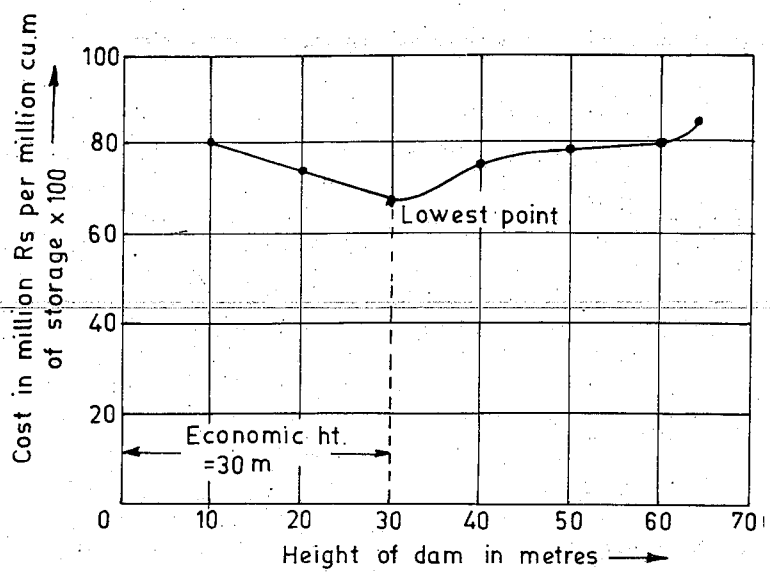


Fig. 18.31.

PROBLEMS

1. (*a*) What is meant by a 'Reservoir'? Discuss briefly the different types of reservoirs and the purpose served by each type.

(*b*) Describe briefly the techniques that are employed for computing the storage capacity of a reservoir for different water surface elevations.

2. (*a*) What is meant by a 'Flood control reservoir' ; and what are their different types ?

(*b*) Discuss with a neat sketch, the various storage zones of the dam reservoir.

(*c*) What factors you will keep in mind while selecting a suitable site for a dam reservoir?

3. (*a*) Differentiate clearly between the following :

(*i*) A flood control reservoir and a multipurpose reservoir.

(*ii*) A retarding basin and a storage reservoir.

(*iii*) Firm yield, design yield, and secondary yield of a reservoir.

(*b*) Briefly describe as to how you would fix the storage capacity of a reservoir and the height of the dam required for this storage. (*Madras University, 1976*)

4. (*a*) What is the relation between 'reservoir capacity' and 'reservoir yield'?

(*b*) How would you fix the capacity of a dam reservoir at a particular river site, provided the inflow pattern and demand pattern are known. Explain the mass curve method which is used for this purpose.

5. (*a*) Explain how the storage capacity of a reservoir is fixed.

(*Madras University, 1973, 1974*)

(*b*) Explain the mass curve method that can be used for determining :

(*i*) Reservoir capacity for fulfilling given demand.

(*ii*) Demand rate from a reservoir of a given capacity.

6. Discuss briefly and with necessary neat sketches, the demand patterns for the following types of reservoirs :

(*i*) Single purpose conservation reservoir.

(*ii*) Single purpose flood control reservoir.

(*iii*) Multipurpose reservoir.

7. (*a*) What is meant by 'flood routing through reservoirs'?

(*b*) Describe step by step procedure that you will adopt for flood routing computations required for reservoirs under 'trial and error method'.

8. Describe a method for routing flood water through a deep reservoir using the fundamental relation between inflow, outflow (discharge) and storage. Take $Q = \text{C.L.}\, H^{3/2}$ for the spillway.

(*U.P.S.C.,Engg. Services, 1974*)

9. The initial inflow in a reservoir and outflow over the spillway was 30 m^3/sec. During a storm, the following inflow rates were noted at the ends of successive half day periods : 240, 300, 240, 90, 30 and 30 m^3/sec. The relationship between discharge over the spillway (Q), and storage rate in the reservoir $\left(\frac{S}{T}\right)$ may be expressed by the equation

$$Q = \frac{1}{2} \cdot \frac{S}{T}.$$

Assuming that the average inflow rate during each half day period is equal to the average of the rates occurring at the beginning and at the end of the period, find the outflow at the end of half hour period.

Hence plot to a suitable scale the inflow and outflow hydrographs.

10. What are the factors on which the rate of silting of an impounding reservoir depends? What is trap efficiency?

Discuss the principal measures that should be undertaken to control the inflow sediment to an impounding reservoir.

An impounding reservoir had original storage capacity for 738 ha-m. The drainage area of the reservoir is 80 sq. km, from which, annual sediment discharges into the reservoir at the rate 0.1153 ha-m

per sq. km. of the drainage area. Assuming the trap efficiency as 80 per cent, find the annual capacity loss of the reservoir in per cent per year. *(U.P.S.C., Engg. Services, 1969)*

[**Ans.** 10%]

11. Write short notes on any four of the following :

(*i*) Reservoir losses (*ii*) Reservoir clearance

(*iii*) Economic height of a dam

(*iv*) Cost benefit considerations in planning dam reservoirs.

[**Note.** Please See Chapter 20 of "Hydrology and Water Resources Engineering"]

(*v*) Reservoir sedimentation and its control

(*vi*) Density currents

(*vii*) Trap efficiency

(*viii*) Estimating the life of a reservoir.

19

Design and Construction of Gravity Dams

19.1. Definition, etc.

A gravity dam has been defined as a structure which is designed in such a way that its own weight resists the external forces. This type of a structure is most durable and solid, and requires very little maintenance. Such a dam may be constructed of masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed. They can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the dam. Such a dam is generally straight in plan, although sometimes, it may be slightly curve. The line of the upstream face of the dam, or the line of the crown of the dam if the upstream face in sloping, is taken as the reference line for layout purposes, etc. and is known as the *Base line of the dam* or the '*Axis of the Dam*'. When suitable conditions are available, such dams car be constructed up to great heights. The highest gravity dam in the world is Grand Dixence Dam in Switzerland (284 m), followed by Bhakra dam in India (226 m); both are of concrete gravity type. The ratio of base width to height of all these structures is less than 1 : 1.

19.2. Typical Cross-section

A typical cross-section of a concrete gravity dam is shown in Fig. 19.1. The upstream face may by kept throughout vertical or partly slanting for some of its length, as shown. A drainage gallery is provided in order to relieve the uplift pressure exerted by the seeping water.

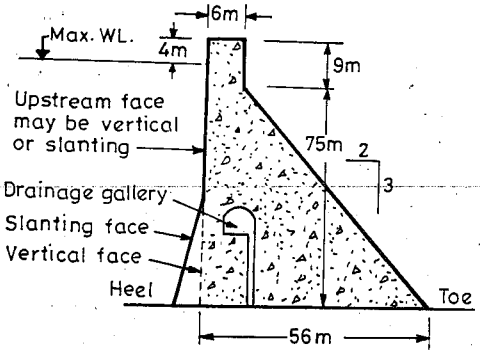


Fig. 19.1. A typical cross-section of a concrete gravity dam.

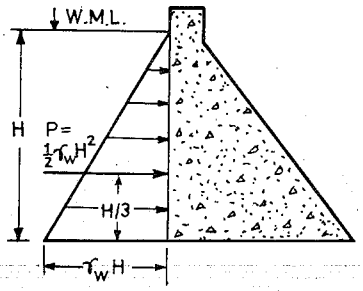
19.3. Forces Acting on Gravity Dam

The various external forces acting on a gravity dam may be :

(1) Water Pressure
(2) Uplift Pressure
(3) Pressure due to earthquake forces
(4) Silt Pressure
(5) Wave Pressure
(6) Ice Pressure
(7) The stabilising force is the weight of the dam itself.

An estimation and description of these forces is given below :

(1) **Water Pressure.** Water pressure (P) is the most major external force acting on such a dam. The horizontal water pressure, exerted by the weight of the water stored on the upstream side on the dam can be estimated from rule of hydrostatic pressure distribution ; which is triangular in shape, as shown in Fig. 19.2 (a) and (b). When the upstream face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base ; where γ_w is the unit weight of water and H is the depth of water : as shown in Fig. 19.2 (a). The resultant force due to this external water $= \frac{1}{2}\gamma_w H^2$, acting at $H/3$ from base.



Where γ_w = unit weight of water
9.81 kN/m^3 = 1000 kgf/m^3
Fig. 19.2. (a)

When the upstream face is partly vertical and partly inclined [Fig. 19.2 (b)], the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v). The horizontal component $P_h = \frac{1}{2}\gamma_w H^2$ acts at $\frac{H}{3}$ from the base ; and the vertical component (P_v) is equal to the weight of the water stored in column $ABCA$ and acts at the c.g. of the area.

Similarly, if there is tail water on the downstream side, it will have horizontal and vertical components, as shown in Fig. 19.2. (b).

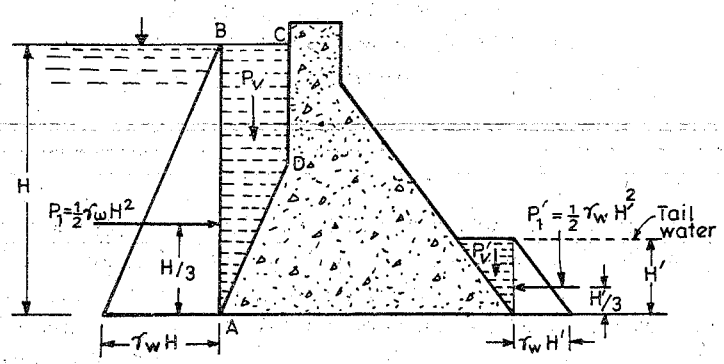


Fig. 19.2 (b)

(2) **Uplift Pressure.** Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through dam body and then to the bottom through the joints between the body of the dam and its foundation at the base ; exert an uplift pressure on the base of the dam. It is the second major external force and must be accounted for in all calculations. Such an uplift force virtually reduces the downward weight of the body of the dam and hence, acts against the dam stability.

The amount of uplift is a matter of research and the present recommendations which are followed, are those suggested by United States Bureau of Reclamation (U.S.B.R.). According to these recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by a

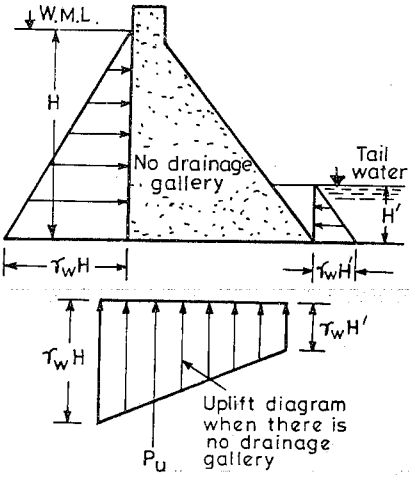


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

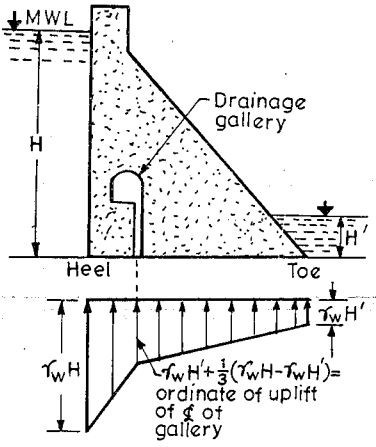


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

straight line in between, as shown in Fig. 19.3 (*a*). When drainage galleries are provided to relieve the uplift, the recommended uplift at the face of the gallery is equal to the hydrostatic pressure at toe ($\gamma_w \cdot H'$) plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe ; as shown in Fig. 19.3 (*b*) ; *i.e.* $\left[\gamma_w \cdot H' + \frac{1}{3}(\gamma_w \cdot H - \gamma_w \cdot H')\right]$. It is also assumed that the uplift pressures are not affected by the earthquake forces.

The uplift pressures can be controlled by constructing cut-off walls under the upstream face, by constructing drainage channels between the dam and its foundation, and by pressure grouting the foundation.

(3) **Earthquake Forces.** If the dam to be designed, is to be located in a region which is susceptible to earthquakes, allowance must be made for the stresses generated by the earthquakes.

An earthquake produces waves which are capable of shaking the Earth upon which the dam is resting, in every possible direction.

The effect of an earthquake is, therefore, equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction, and for design purposes, it has to be resolved in vertical and horizontal components. Hence, two accelerations, *i.e.* one horizontal acceleration (α_h) and one vertical acceleration (α_v) are induced by an earthquake. The values of these accelerations are generally expressed as percentage of the acceleration due to gravity (g), *i.e.* $\alpha = 0.1\ g$ or $0.2\ g$, etc.

In India, the entire country has been divided into five seismic zones depending upon the severity of the earthquakes. Zone V is the most serious zone and includes Himalayan regions of North India. A map and description of these zones is available in "Physical and Engineering Geology" (1999 edition) by the same author, and can be referred to, in order to obtain an idea of the value of the α which should be chosen for designs. On an average, a value of α equal to 0.1 to 0.15 g is generally sufficient for high dams in seismic zones. A value equal to 0.15 g has been used in Bhakra dam design, and 0.2 g in Ramganga dam design. However, for areas not subjected to extreme earthquakes,

$\alpha_h = 0.1\ g$ and $\alpha_v = 0.05\ g$ may by used. In areas of no earthquakes or very less earthquakes, these forces may be neglected. In extremely seismic regions and in conservative designs, even a value upto 0.3 g may sometimes be adopted.

Effect of vertical acceleration (α_v). A vertical acceleration may either act downward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body ; thus reducing the effective weight and the stability of the dam, and hence is the worst case for designs.

Such acceleration will, therefore, exert an **inertia force** given by

$$\frac{W}{g}\alpha_v \text{ (}i.e.\text{ force = Mass} \times \text{Acceleration)}$$

where W is the total weight of the dam.

$\therefore$ The net effective weight of the dam $= W - \frac{W}{g} \cdot \alpha_v$.

If $\alpha_v = k_v \cdot g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam

$$= W - \frac{W}{g} \cdot k_v \cdot g = W[1 - k_v].$$

In other words, vertical acceleration reduces the unit weight of the dam material and that of water to $(1 - k_v)$ times their original unit weights.

Effects of horizontal acceleration (α_h). Horizontal acceleration may cause the following two forces :

(*i*) Hydrodynamic pressure ; and

(*ii*) Horizontal inertia force.

Both these forces are discussed below :

(*i*) *Hydrodynamic pressures.* Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure, as the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as *hydrodynamic pressure.*

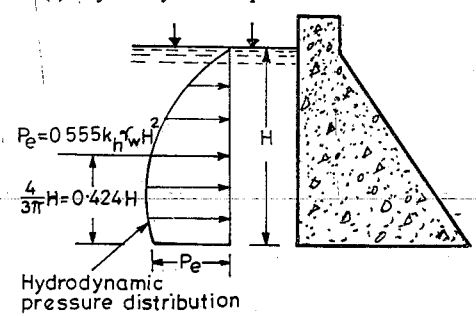


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

According to **Von-Karman**, the amount of this hydrodynamic force (P_e) is given by.

$$P_e = 0.555 \cdot k_h \gamma_w \cdot H^2 \qquad ...(19.1)$$

and it acts at the height of $\frac{4H}{3\pi}$ above the base, as shown in Fig. 19.4.

(2) Rook-fill dams

(3) Solid masonry gravity dams.

These types are discussed below :

(1) **Earth Dams.** Earth dams are made of soil that is pounded down solidly. They are built in areas where the foundation is not strong enough to bear the weight of a concrete dam, and where earth is more easily available as a building material compared to concrete or stone or rock.

Some important earth dams of the world are :

(*i*) Green mountain dam on Colorado river in U.S.A.

(*ii*) Swift dam in Washington in U.S.A.

(*iii*) Side flanks of Nagarjun Sagar dam in India.

(*iv*) Trinity Dam in California in U.S.A.

(*v*) Maithan Dam in India (which is partly Earthen and partly Rockfill).

(2) **Rockfill Dams.** Rockfill are formed of loose rocks and boulders piled in the river bed. A slab of reinforced concrete is often laid across the upstream face of a rockfill dam to make it water-tight.

Some important rock-fill dams of the world are ;

(*i*) The Salt Springs Dam in California (345′ high) in U.S.A.

(*ii*) The San Gabriel No. 1 Dam (321′ high) in U.S.A.

(*iii*) Cougar Dam on Mc-Knezie River in Oregon (445′ high) in U.S.A.

(3) **Solid-masonry Gravity dams.** These are familiar to us by now, after we have talked about Aswan, Roosevelt, Hoover, and above all Bhakra dam.

These big dams are expensive to be built but are more durable and solid than earth and rock dams. They can be constructed on any dam site, where there is a natural foundation strong enough to bear the great weight of the dam.

These three types of dams were all found in ancient days. In recent times, *four other types of dams have come into practice.* They are :

(4) Hollow masonry gravity dams ;

(5) Timber dams ;

(6) Steel dams ; and

(7) Arch dams.

(4) **The hollow masonry gravity dams.** These are essentially designed on the same lines on which the solid masonry gravity dams are designed. But they contain less concrete or masonry ; about 35 to 40% or so. Generally, the weight of water is carried by a deck of R.C.C. or by arches that share the weight burden. They are difficult to build and are adopted only if very skilled labour is easily available, otherwise the labour cost is too high to build its complex structure.

(5) **Steel dams.** These are not used for major works. Today, steel dams are used as temporary coffer dams needed for the construction of permanent dams. Steel coffer dams are usually reinforced with timber or earthfill.

(6) **Timber dams.** These are short lived, since in a few years time, rotting sets in. Their life is not more than 30 to 40 years and must have regular maintenance during that time. However they are valuable in agricultural areas, where a cattle raiser may need a pool for his live stock to drink from, and for meeting other such low-level needs.

(7) **Arch dams.** Arch dams are very complex and complicated. They make use of the horizontal arch action in place of weight to hold back the water. They are best suited at sites where the *dam must be extremely high and narrow.* Some examples are :

(*i*) *Sautet dam* on the Drac River in France, 414′ high, but only 230′ long at top and 85′ long at bottom of the gorge, 56′ thick at bottom and 8′ thick at top.

(*ii*) *The Tignes dam* in France (592′ high).

(*iii*) *Mauvoisin dam* on the Drause River in Switzerland, (780′ high).

(*iv*) *Idduki* dam in Kerala State, across the Periyar river, which is the only arch dam in India. It is 366 m (1200′) long double curvature arch dam, made in concrete, and has a height of about 170 m (560′).

17.3. Problems in Dam Construction

Dams are extremely useful things. Anyone who lives in Punjab or at Asansol in West Bengal, knows how valuable dams are. The farmers of Punjab and people getting electricity from the Bhakra sing praises for it. The people of areas benefitted by various dams and other ancillary works on Damodar river are really thankful to those human beings who have miraculousy harnessed the Damodar river for them. The prosperity and welfare of millions and billions of people depend directly on these towering handsome dams with which the nation's rivers have been harnessed.

But dams can cause problems too. Dams have drawbacks and disadvantages also. Let us here discuss some of the negative features of dams and let us see what can be done to overcome them. There are four major problems, in general, which are posed by such huge constructions. They are:

(1) Fish Problem ;

(2) Submergence Problem ;

(3) Failure Problem; and

(4) Bomb Problem

They are described below:

(1) **Fish Problem.** On large rivers, in late summer season, fish move from downstream to upstream to lay their eggs. These eggs are fertilised by male fish. The old fish may get exhausted and the new born fish again move downstream. They, after two to three years, return to their ancestral spawning place and may die after getting exhausted, while the newborns move downstream. The cycle goes on for years.

The fish which move to their ancestral spawning place (upstream) are called *anadromous fish. Salmon and Hilsa* are typical examples of such a fish. These are commercially valuable fish, and important industries are dependent on them.

When a dam barrier is constructed on a river, these fish can not move upstream to lay their eggs; because it is impossible for these fish to overtop such a barrier. But surprisingly, even when they find a barrier in their path of advancement towards their ancestral spawning ground, these fish do not return to their downstream dwelling place (*i.e.* sea). However, they go on fighting against the barrier, trying furiously to overtop it, till they get exhausted and die down. This results in a serious large scale killing of fish, causing great damage to fish industry and economy of the nations.

In the beginning, much attention was not paid to this problem; but a little later, it was realised, and serious attempts were made to find out solution to the problem.

Sometimes, fish were trapped on one side of the dam and passed on to the other side by giant steel and plastic nets. An external arrangement called *Fish Ladder* was also devised.

Fish Ladder. Just as river-going vessels can bypass a dam by using a navigation lock, so a series of 'locks' enable the fish to get over the dam. A separate channel is created, consisting of a series of little dams that form a row of pools, rising up over the big dam to reservoir level. The salmon, entering the lowest rung of the ladder at the base of the dam, could leap from pool to pool until they had crested the dam. Then, they could continue on through the reservoir to the spawning grounds. The new born fish called *finger lings* could later return to the sea (downstream) in the same fashion via the ladder. A section, plan and photographic view of a fish ladder has already been shown in the chapter on Weirs.

In the beginning, the fish ladders worked better in theory than in practice. The fish seemed to prefer to mill ground in splashing water under spillway, instead of entering the ladder. This difficulty was overcome by careful design that put the fish ladder in the place where it was most likely to attract the fish. Another problem was that the slow moving water was stranger to fish and they tended to collect in the lower pools without going onward.

Millions and billions were spent into fish-ladder research. Improvements in design made the fish ladder more attractive to fish, more like the rapids they were accustomed to.

Fish ladders are not always practicable from engineering stand point. In such cases, other steps have to be taken to protect the fish.

Meanwhile, other experiments are going forward to see if fish can be successfully induced to spawn in waters other than their own ancestral spawning grounds. In the long run, it may save millions of currency to construct *fish hatcheries* instead of *fish-ladders.* There are many possible solutions to the problem of anadromous fish, and research is being undertaken in different regions of the world to find out a better solution to the problem.

(2) **Submergence Problem.** Whenever a dam is constructed across a river to store water on the upstream side, a large area gets submerged due to the rise in the water levels. The entire area which gets submerged, forming a reservoir, has to be calculated and acquired before a dam can be constructed. The owners of the land have to be persuaded, adequately compensated, and well settled somewhere else, before, the work can be taken up in hand. Hence it is necessary to investigate the probable damage caused by this submergence.

(3) **Failure Problem.** We try our best to build dams to last as long as possible. Every person whosoever has worked on a dam hopes that the dam will live as long as the pyramids of Egypt. But many a times, the dam give way under the continued insistent pressure of the water penned up behind them. This failure of the dam may be caused either due to bad workmanship or due to faulty design or due to the occurrence of unanticipated floods.

Luckily, these disasters have been comparatively rare in this century. Dams used to give way easily in olden times, but due to engineering advancement in modern times, their failure has been considerably reduced.

These huge structures are now properly designed, keeping in view the various forces which they are going to face. Proper and rational design, good supervision and constant vigil and watch during maintenance period ensures their safety and makes us fairly confident of it. Bhakra Dam on Satluj River in India and Boulder Dam on Colorado River in U.S.A. *cannot fail* in one attempt, how furiously these rivers may try to move their foundations. We are fairly confident of this, but sometimes the confidence is *rudely and cruelly repaid with tragedies.*

Dams may sometimes fail due to excessive and unanticipated earthquakes. The *Koyna Dam* in India was at the verge of failure in 1968 earthquake. Thanks to the efforts of the Indian engineers who saved that dam by toiling hard day and night. A very confident dam called *Vega de Tera Dam* is Spain failed in January 1959. The people were tucked in the town of Rivaldelago. The disaster caused was tremendous. Rivaldelago was flattened. Telephone poles were snapped like matchsticks. Within moments, 123 villages were drowned. Several hundred luckier ones were saved, but were rendered homeless. This was a case where a dam had simply not been built strong enough to bear the full weight of its intended reservoir. Heavy rains wrecked it. *Faulty design and bad engineering must be blamed.*

Another important dam called *The Malpasset Dam,* a 200 feet high arch dam on the Reyran River, was completed in 1954. This dam gave way in December 1956, causing 421 persons to die in floods. Investigations revealed that the dam had failed because the foundation rock has shifted along a thin clay seam in the left abutment, making the dam unstable and vulnerable to any serious stress.

We learnt from our mistakes;and several other dams of the same type, then under construction in Europe, were quickly resurveyed to find the possibility of such a geological formation. This was very very small comfort to the relatives of those who died when Malpasset failed; but at least, we should learn from our mistakes and there should be no such repetitions.

(4) **The Bomb Problem.** The dams create dangers in wars, especially in modern atomic age. One single atom bomb may cause the failure of Hoover Dam (Boulder Dam) or Bhakra Dam. The resultant failure of such a dam will create catastrophes, but also, it will get contaminated by radioactivity from which there could be no escape.

This is an important point which is generally stressed by opponents of big dams. But the only answer to this argument is that it would not be advisable to deprive ourselves of the benefits of big dams simply because they are hazards in war time. After all, an atom bomb dropped in Calcutta, Delhi, or New York would also cause tremendous damage and catastrophe, but this *does not mean* that we should not *develop big cities.*

Atomic war is dangerous to every aspect of living and not only to the construction of dams. We don't refuse riding in automobiles or aeroplanes because of the fear of accidents. *Certain risk has to be accepted if there is to be progress.*

So, without denying the very great damage that could be caused by atomic explosions at our dams, we must go on building dams. We need them and we must devote our energies to the cause of continued peace, so that bombs will never be able to fall. We may also take more precautions, and anticraft guns and radars can be established at and in the vicinity of such important works. The use of atomic energy for peaceful purposes and a general feeling of brotherhood is the only possible way to reduce such threats.

17.4. Selection of the Type of Dam and Their Classifications

(1) *Classification.* Dams can be classified in various ways depending upon the purpose of the classification.

(1) *Classification According to the Material used for Dam Construction :*

The dams classified according *to the material used* for construction are: Solid masonry gravity dams, Earthen dams, Rockfill dams, Hollow masonry gravity dams, Timber dams, Steel dams, and R.C.C. Arch dams. They have already been explained in a previous article.

(2) *Classification According to Use*

(*i*) **Storage Dams.** They are constructed in order to store water during the periods of surplus water supply, to be used later during the periods of deficient supply. The stored water may be used in different seasons and for different uses. They may be further classified depending upon the specific use of this water, such as navigation, recreation, water supply, fish, electricity, etc.

(*ii*) **Diversion Dams.** These small dams are used to raise the river water level, in order to feed an off-taking canal and or some other conveyance systems. They are very useful as irrigation development works. A diversion dam is generally called a **weir** or a **barrage.**

(*iii*) **The Detention Dams.** They detain food-waters temporarily so as to retard flood runoff and thus minimise the bad effects of sudden flood.

Detention dams are sometimes constructed to trap sediment. They are often called debris dams.

(3) *Classification According to Hydraulic Designs*

(*i*) **Overflow Dams.** They are designed to pass the surplus water over their crest. They are often called Spillways. They should be made of materials which will not be eroded by such discharges.

(*ii*) **Non-overflow Dams.** They are those which are not designed to be overtopped. This type of design gives us wider choice of materials including earthfill and rockfill dams.

Many a times, the overflow dam and the non-overflow dam are combined together to form a composite single structure.

(*iii*) **Rigid Dams and Non-rigid Dams.** Rigid dams are those which are constructed of rigid materials like masonry, concrete, steel, timber, etc.; while non-rigid dams are constructed of earth and rock-fill. They have already been explained.

17.5. Factors Governing the Selection of a Particular Type of Dam

Whenever we decide to construct a dam at a particular place, the first baffling problem which faces us, is to choose the kind of the dam. *Which type will be the most suitable and most economical?* Two, three kinds of dams may be technically feasible, but only one of them will be the most economical. Various designs and their estimates have to be prepared before signalling one particular type. The various factors which must be thoroughly considered before selecting one particular type are described below:

(1) **Topography.** Topography dictates the first choice of the type of dam. For example:

(*i*) A narrow U-shaped valley, *i.e.* a narrow stream flowing between high rocky walls, would suggest a concrete overflow dam.

(*ii*) A low, rolling plain country, would naturally suggest an earth fill dam with a separate spillway.

(*iii*) The availability of a 'Spillway Site' is very important while selecting a particular kind of a dam.

(*iv*) A narrow V-shaped valley indicates the choice of an arch dam. It is preferable to have the top width of the valley less than one-fourth of its height. But a separate site for the spillway must also be available.

(2) **Geology and Foundation Conditions.** The foundations have to carry weight of the dam. The dam site must be thoroughly surveyed by geologists, so as to detect the thickness of the foundation strata, presence of faults, fissured materials, and their permeability, slope, and slip, etc.

The various kinds of foundations generally encountered are discussed below:

(*i*) *Solid Rock Foundations.* Solid rock foundations such as granite, gneiss, etc. have a strong bearing power. They offer high resistance to erosion and percolation. Almost every kind of dam can be built on such foundations. Sometimes, seams and fractures are present in these rocks. They must be grouted and sealed properly.

(*ii*) *Gravel Foundations.* Coarse sands and gravels are unable to bear the weight of high concrete gravity dams and are suitable for earthen and rock-fill dams. Low concrete gravity dams up to a height of 15 m may also be suggested on such foundations.

These foundations have high permeability and, therefore, subjected to water percolation at high rates. Suitable cut-offs must be provided to avoid danger of undermining.

(*iii*) *Silt and Fine Sand Foundations.* They suggest the adoption of earth dams or very low gravity dams (upto height of 8 m). A rockfill dam on such a foundation is not suitable. Seepage through such a foundation may be excessive. Settlement may also be a problem. They must be properly designed to avoid such dangers. The protection of foundations at the downstream toe from erosion must also be ensured.

(*iv*) *Clay Foundations.* Unconsolidated and high moisture clays are likely to cause enormous settlement of the dam. They are not fit for concrete gravity dams or for rock-fill dams. They may be accepted for earthen dams, but that too, after special treatment. (*v*) *Non-uniform Foundations.* At certain places, a uniform foundation of the types described above may not be available. In such a case, a non-uniform foundation of rock and soft material may have to be used if the dam is to be built. Such unsatisfactory conditions have to be dealt with by special designs. However, every problem is an individual problem and a solution has to be found by experienced engineers. For example—

The Jawahar Sagar Dam in Rajasthan offered such a problem. A bed of clay was encountered, between the base of the dam and solid rock foundation. It was not economically feasible to remove this clay bed. The solution adopted was to anchor the base of the dam to the foundations below, by means of prestressed cables.

(3) **Availability of Materials.** In order to achieve economy in the dam, the materials required for its construction must be available locally or at short distances from the construction site.

Sometimes, good soil is easily available, which naturally calls for an earthen dam. If sand, cement and stone, etc., are easily available, one should naturally think of a concrete gravity dam. If the material has to be transported from far off distances, then a hollow concrete dam (Buttress) is a better choice.

(4) **Spillway Size and Location.** Spillway, as defined earlier, diposes of the surplus river discharge. The capacity of the spillway will depend on the magnitudes of the floods to be by- passed. The spillway will, therefore, become much more important on streams with large flood potential. On such rivers, the spillway may become dominant structure, and the type of dam may become the secondary consideration.

The cost of constructing a separate spillway may be enormous or sometimes a suitable separate site for a spillway may not be available. In such cases, combining the spillway and the dam into one structure may be desirable, indicating the adoption of a concrete overflow dam.

At certain places, where excavated material from a separate spillway channel may be utilised in dam embankment, an earthfill dam may prove to be advantageous. Small spillway requirement often favours the selections of earth fill or rockfill dams even in narrow dam sites.

The practice of building a concrete spillway on earth and rock embankments is being discouraged these days, because of their conservative design assumptions and the vigil and watch that has to be kept during their operations.

(5) **Earthquake Zone.** If the dam is to be situated in an earthquake zone, its design must include the earthquake forces. Its safety should be ensured against the increased stress induced by an earthquake of worst intensity. The type of structures best suited to resist earthquake shocks without danger are earthen dams and concrete gravity dams.

(6) **Height of the Dam.** Earthen dams are usually not provided for heights more than 30 m or so. Hence, for greater heights, gravity dams are generally preferred.

(7) **Other Considerations.** Various other factors such as, the life of the dam, the width of the roadway to be provided over the dam, problem of skilled labour, legal and aesthetic point must also be considered before a final decision is taken. Overall cost of construction and maintenance and the funds available will finally decide the choice of a particular kind of a *dam at a particular place.*

17.6. Selection of Dam Site

The selection of a site for constructing a dam should be governed by the following factors:

(1) Suitable foundations (as determined in the previous article) must be available.

(2) For economy, the length of the dam should be as small as possible, and for a given height, it should store the maximum volume of water. It, therefore, follows, that the river valley at the dam site should be narrow but should open out upstream to provide a large basin for a reservoir. A general configuration of contours for a suitable site is shown in Fig. 17.1.

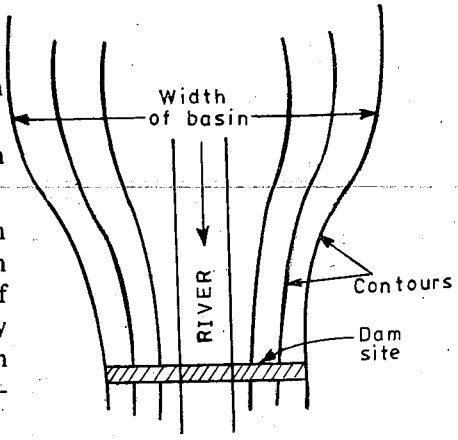


Fig. 17.1

(3) The general bed level at dam site should preferably be higher than that of the river basin. This will reduce the height of the dam and will facilitate the drainage problem.

(4) A suitable site for the spillway should be available in the near vicinity. If the spillway is to be combined with the dam, the width of the gorge should be such as to accommodate both.

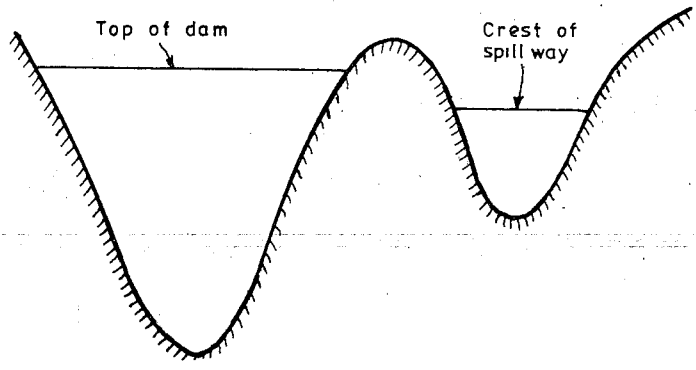


Fig. 17.2

The best dam site is one, in which a narrow deep gorge is separated from the flank by a hillock with its surface above the dam, as shown in Fig. 17.2.

If such a site is available, the spillway can be located separately in the flank, and the main valley spanned by an earthen or similar dam. Sometimes, the spillway and concrete masonry dam may be compositely spanned in the main gorge, while the flanks are in earth at low cost.

(5) Materials required for the construction should be easily available, either locally or in the near vicinity, so that the cost of transporting them is as low as possible.

(6) The reservoir basin should be reasonably water-tight. The stored water should not escape out through its side walls and bed.

(7) the value of land and property submerged by the proposed dam should be as low as possible.

(8) The dam site should be easily accessible, so that it can be economically connected to important towns and cities by rails, roads, etc.

(9) Site for establishing labour colonies and a healthy environment should be available in the near vicinity.

STORIES OF A FEW IMPORTANT DAMS

Before we take up the actual planning and design of concrete gravity dams and earthen dams in subsequent chapters, let us narrate the stories of certain such important dams. This will give us an idea as to what actually happens in the field and to what kind of difficulties are encountered and how they are overcome.

17.7. Hoover Dam

Hoover dam (Fig. 17.3) is a concrete gravity dam, constructed on the Colorado river in California (U.S.A.). The construction of this dam was taken off the drawing boards on January 26, 1892 when the Colorado River Commission presided by Mr. Harbert Hoover discussed its construction in their first meeting. But the actual construction of this dam could start only in late 1930. The construction took about 2 years of non-stop work, every minute of the hour, 24 hours a day, and 365 days a year. On the day of Christmas, the pouring of concrete was tremendous even under searing desert sun by day and under floodlights at night.

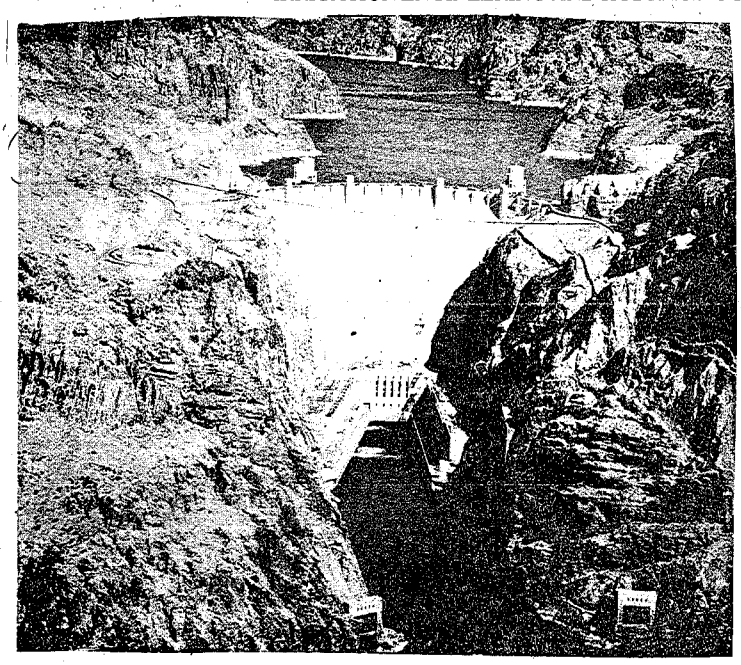


Fig. 17.3. Photo view of Hoover Dam.

Name Problem. The name of the Hoover Dam became a political issue. It was first of all named as *Boulder Dam.* Later when Herbert Hoover became the President of America, its name was changed to *Hoover Dam.* In 1933, when the Democrats replaced the Republicans, President Roosevelt changed this name again to *Boulder Dam.* It was known as *Boulder Dam* for 14 years. When again in 1946, Republicans came to power, the name was once again changed to *Hoover Dam,* which still exists today.

Dimensions of the Dam. The original dimensions of the dam are given below in F.P.S. units:

Height	= 726 ft
Span	= 1300 ft from rim to rim of the Canyon
Thickness at the base	= 660 ft
Thickness at the top	= 45 ft
Name of the reservoir formed at the back of the dam	= Lake Mead
Length of the reservoir	= 115 miles.

Something about the river. This big dam straddles the Colorado river. The Colorado river rises in the State of Colorado, runs down through Utah and into Arizona, then California, emptying finally into the Gulf of California in Mexico.

The Colorado is a river with muscles. It is a very strong river - containing huge amounts of silt and mud in it. It cuts numerous deep narrow canyons while it flows. The mile deep walls of the Grand canyon proves the power of this river.

It is a young river and all such rivers have strong powers for forming deep narrow gorges (*i.e.* canyons). They are the most turbulent, and the deepest digging. They are the best sources of hydroelectric power. Irrigation or navigation is not possible on this kind of rivers, unless, they are tamed.

Attempts were made to tame this river by constructing dikes or levees, but failed.

Spring floods of 1904 brought havoc. Another catastrophe occurred in 1916. Gila River, a tributary of Colorado, flooded at a rate of about 5, 560 cubic metres/sec. The town of Yuma in Arizona, where the Gila empties into the Colorado, was submerged to a depth of 1.2 metres.

It was extremely desirable to tame this river. But how? A solution was dreamt of. The idea of constructing a big dam was visualised by Arthur Davis. It was an ambitious plan.

Planning for the Dam. A scheme was planned. A concrete gravity dam, of about 700 to750 feet height was thought. A reservoir at the back of the dam could hold every drop of water, the Colorado could send in any 2 years of steady flow. It was to be the biggest man-made lake in the world. It was going to be a multi- purpose dam, generating about 6 billion Kilo watt-hours of electrical energy each year for the growing cities of southern California. The reservoir would hold the flood waters, and the spillways would release the required amounts of water to the downstream farmers.

Flood control, irrigation and electricity were the three main purposes of this project. Supply of drinking water to 13 cities of California and creation of a recreational and navigable lake reservoir were the additional advantages.

Selection of a suitable site. A thorough search was made for the spectacular gorges of the Colorado, seeking the best possible site. They studied 70 such sites before choosing 'Black canyon' on the border between Arizona and Nevada, 48 km from the city of Las Vegas (Nevada).

The preliminary survey of *Black canyon* took about 3 years. Here, the river flows through cliffs 1000-2000 feet (300 m to 600 m) high. At the water line, the rock walls were 350 feet (105 m) apart.

Engineers roamed in heat and sun, testing the rocks, drilling into it, to make sure that it could stand the burden of enormous weight of the concrete that would be laid upon it. *Their conclusion was that it could. They recommended that the giant dam be built.*

After the technical green-signal was obtained, some political issues such as to who will be benefitted and up to what extent, were settled with a great difficulty. The rift between the different States always persists in such huge projects.

Construction of the Dam. The real work began in late 1930. Herbert Hoover was the President of America at that time. He himself was an engineer. The work started under his vigilance, Arthur Powell Davis, 70 years old and about to retire, saw his life long dream fulfilled as he stood high above the Colorado and watched thousands of workers working hard with picks and dynamite far below.

The dam site was very hot. It was not a congenial surroundings to work. But a *dam had to be built.*

First of all, a town was built to house the workers. A permanent city, now called Boulder-city, was settled at an expenditure of about $70 million. This was spread in an area of about 300 acres and could accommodate 5, 000 workers.

Whenever a dam is constructed, the water of the river is first of all to be diverted so that the construction could start. This is called by engineers as 'to turn the river off.' 4 tunnels, each 56 feet wide and 4000 feet long were dug into the solid rock of the canyon walls. These *bypass tunnels* received the flow of the river and carried it down

to the downstream, to a point beyond the construction site. About 1 $\frac{1}{2}$ million cubic feet of rock had to be removed for building these four tunnels.

Then, the Coffer dam was built. A coffer dam is a temporary retaining embankment upstream of the site. Huge amounts of rock and earth was heaped up, forcing the river into four bypass tunnels. The bed of the river was thus laid bare.

Workers then descended into this river bed to lay the foundations for the dam. Seven million tons of concrete had to be laid. It was 660 feet thick at the base.

The dam curves upstream, so that the water load is held back in part by the walls of the canyon. The completion of the dam took about 2 years of non-stop work.

After completion, the bypass tunnels were blocked up and the water started coming and collecting against the dam. The lake formed on the upstream side was called *Lake mead.* Electrical power houses, which are of the size of 20-storey sky-scrapers were constructed. The lake, the power houses, the dam galleries etc. are open for visitors and for inspection. It was a great achievement indeed.

17.8. Bhakra Dam

Bhakra Dam is a concrete gravity dam. It is 740 feet **(226 m)** high, spanning the V-shaped gorge in the lower Shivalik hills. The dam is 1700 ft long at the top and only 325 ft at the bottom. The thickness of the dam at foundations is 1320 ft and it tapers to 30 ft at the top where a road runs. *Bhakra dam was the highest concrete gravity dam of the world when built, thus surpassing the existing* 726 ft (221 m) high Hoover dam. But the highest concrete gravity dam of the world, at present, is **Grand Dixence dam** in Switzerland **(284 m high).** Bhakra dam is situated in Himachal Pradesh State of India near a village used to be called Bhakra. It has been constructed on Satluj river. Satluj is a river coming from Himalayas. It is a perennial river but carries enormous water during floods and rains.

Downstream and upstream views of Bhakra dam are shown in Photo Fig. 17.4.

This dam has given tremendous prosperity to India and has given her a high name in the world. The various functions served by this dam are:

(*i*) Flood control (*ii*) Irrigation

(*iii*) Electricity (*iv*) Fish development.

Bhakra Project is not a single Bhakra Dam but consists of the following:

(*i*) 740 ft high Bhakra Dam (*ii*) 95 ft high Nangal Dam

(*iii*) Nangal Hydel Channel (*iv*) Ganguwal and Kotta Power houses

(*v*) Bhakra Canal System.

Planning and Construction. The survey works for the construction of this multi-purpose project started in 1919. From 1919 to 1930, the survey continued and various sites were considered for various purposes. From 1932 to 1946, the work was inter-rupted, and finally in 1946, a railway line was spread in this area, and then the actual construction started.

After selecting a suitable narrow canyon for the construction of Bhakra Dam, the dam site was dewatered after the river was 'turned off'. Two diversion tunnels, one in either abutment were constructed in order to carry the river water. Two coffer dams enclosing the foundation area, were also constructed. Both the tunnels are 50 ft, in finished diameter, half a mile long and are lined with 3.6 ft thick heavily reinforced concrete. The work on these two tunnels was started in 1948 and was completed in 1953. The total expenditure incurred on them was approximately Rs. 3.6 crores.

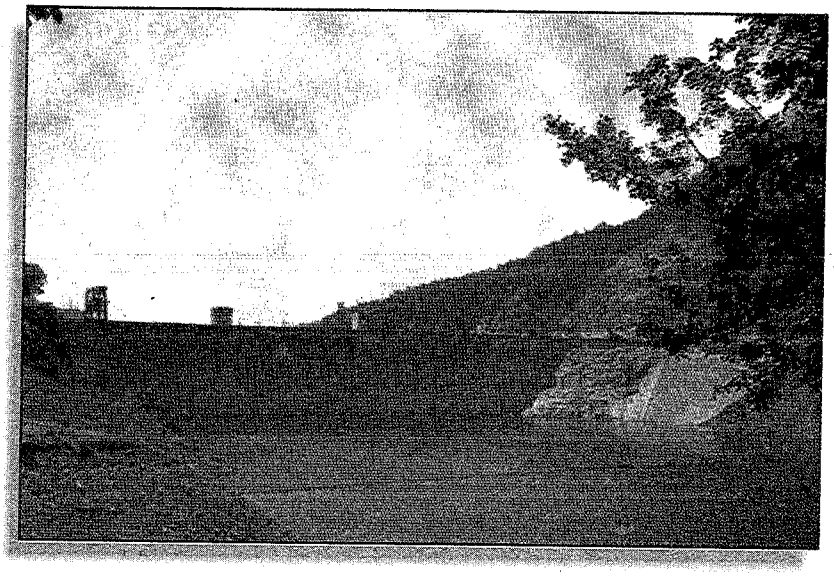


Fig. 17.4 (*a*) Upstream view of Bhakra Dam.

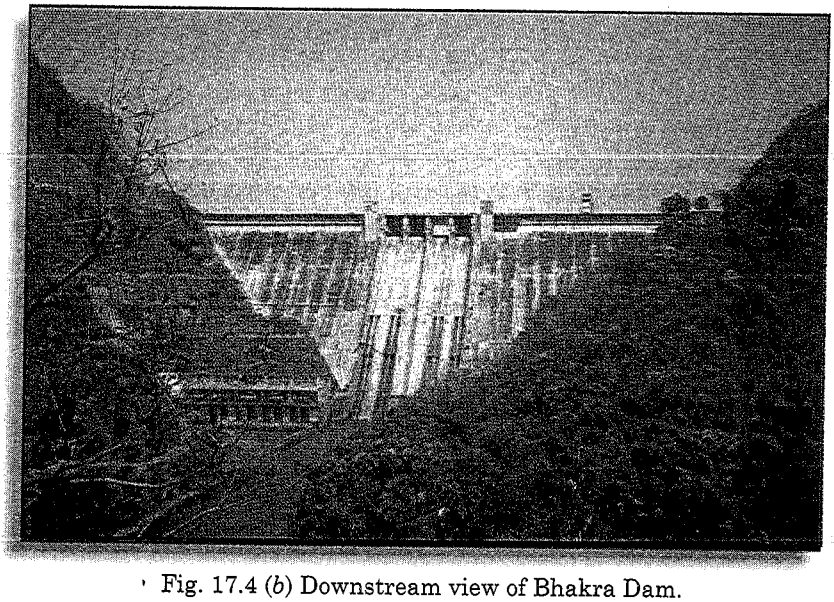


Fig. 17.4 (*b*) Downstream view of Bhakra Dam.

Fig. 17.4. Photoviews of Bhakra Dam.

After the construction of these tunnels, two earth (rolled) and rock coffer dams were constructed to enclose the operation area of the dam site. The construction of 215 ft high coffer dam began in 1956. The water was forced into the tunnels and taken downstream beyond the construction point. The 15 ft high downstream coffer dam did not let the water come back in the pit.

The foundations of these rolled fill earthen dams or dikes were carried down to 60ft and 70 ft below the river bed level. After the construction of coffer dams, the dewatering of the area was expeditiously completed and the excavation of the river bed started. The rock was blasted by an explosive. Desert-shovels were employed for loading the material in special trucks having capacity of 914 cubic yards. The material was carried and dumped at the dumping site. The total excavation was of the order of 700 million cubic yards. About one million cubic yard was excavated per day.

While the excavation was going on, a large constructional plant, capable of producing high quality concrete economically and efficiently was also installed. This required a $4\frac{1}{2}''$ long belt conveyor system, aggregate processing plant, cement handling plant, cooling plant, batching and mixing plants, high steel tristles, revolving cranes and other electricity operated cranes, etc. etc. It was an expensive, completely automised concrete plant. It was capable of handling about *600 tons* of concrete per hour.

Then, the actual concreting work of the dam started in November 1956. Huge electric cranes including 6 cantilever-cranes and 2 stiff-legged-cranes were employed for concreting of dam.

A very low heat cement was used in the construction of this dam. Because, when cement sets, it produces a large amount of heat which is liable to cause cracks in the structure. So in order to avoid this cracking, steel pipes were embedded in the structure and ice-cold water was circulated through them.

The first stage of the dam (390 ft.) was completed by 1959. Work in the 2nd stage was interrupted by a flood in the diversion tunnel that drowned ten workers and damaged the power house. The tunnel and plugging of the dam was completed in 1962.

Dimensions and other data about Bhakra Dam are given below:

Height of the dam	= 740′
Length at the bottom	= 325′
Length at the top	= 1200′
Breadth or thickness at the bottom	= 1320′
Breadth or thickness at the top	= 30′
Concrete required for the dam	= 55 lakh cubic yard
Electricity generated	= 12 lakh Kilowatt
Name of the reservoir formed	= Govind Sagar
Length of the reservoir	= 90 miles
Maximum depth	= 740′
Minimum depth	= 300′
Total irrigation	= 1 crore acre of area
Population of these villages	= 30, 000
Total storage capacity	= 8 million acre ft.
Catchment area	= 22, 000 sq. miles
Live storage	= 6.35 million acre ft.
No. of Inspection galleries	= 46

Power Houses. There are two power houses called (*i*) The Left Power House; and (ii) The Right Power House. The left power house is more important and was first constructed.

The left power house is a reinforced concrete structure. It is standing on a stone consisting partly of clay stone and partly of sand stone. The foundation chosen were adverse to the geological conditions required for a good dam site. The permeability of sand stone is high, and thus there was a possibility of leakage and danger of undermining of the foundations. It necessitated a heavily reinforced raft foundation. The left power house building is seven storeyed and its construction took about two to three years. It required about 635, 000 cubic yards of concrete, about one million sq.ft. of finishing work and 6, 000 tons of reinforced steel.

The machinery used in this power house is of cosmopolitan in nature. The turbines were supplied by Japan, the generators and transformers by U.K., and over-head cranes by Yogoslavia.

The local manufacture of 10 ft diameter penstocks, made out of 1.75″ thick steel plate was considered as a big engineering feat of the country in such a short period of independence. The left power house had costed nearly Rs. 10 crores. The work on right power house was also completed afterwards..

Problem of Wood Transport. Like all other perennial rivers of India, the River Satluj has been the cheapest means of transporting wood from the Himalayan forests down to the plains. But since the dam is very high above the river water level, it has become impossible to use this river for this purpose.

The problem is overcome by bringing logs of woods from Govind Sagar to Nangal railway station by means of an aerial rope-way. This is about $5\frac{1}{2}$ miles long. A wooden log loom has been put across Govind Sagar to obstruct timber. From there, the timber is taken by the inclined carriage way to the loading station on the upstream of the right side.

17.8.1. Nangal Dam. Nangal Dam is 95 ft high subsidiary dam, 8 miles downstream of Bhakra on Satluj. It falls within the jurisdiction of Punjab State in India. The length of the dam is 1, 000 ft. The object of this dam is to head up water of the river Satluj and then divert it into the canal off-taking from the left bank of the river. The canal is called Nangal Hydel Canal, and is a 40 miles long concrete lined canal.

This dam has 29 strong gates of span 30 ft each. An enormous tunnel called the Inspection gallery has been made in the river Satluj in the lower portion of this dam. This was the first tunnel constructed by the Government under Bhakra Project Scheme. In order to enter into this tunnel, one has to go 70 ft down. The tunnel goes across Satluj River.

Bhakra Nangal Project is something tremendous, stupendous, something which shakes up and thrills us when we see it. It marks the India's progress after her Independence. It is something which cannot be forgotten easily, if we see it once.

17.9. Nagarjuna Sagar Dam

The multipurpose Nagarjuna Sagar dam is located across Krishna River, near Nandikonda village in Nalgonda District (Andhra Pradesh). It is named after Buddhist Savant, Acharya Nagarjuna, who lived at the spot about 2,000 years ago to fulfil a mission.

This Dam irrigates in Guntur, Kurnool, Nellore, Nalgonda, Khamman and Krishna Districts. Its irrigation Potential is about 35 lakh acres of land and electrical potential is 1 lakh kilowatt hours of firm power (guaranteed-power generation) and 4.6 lakh kilowatt hours of seasonal power.

A photoview of this dam is shown in Fig. 17.5.

Salient Features of the Dam. The river gorge is blocked by a masonry dam 409 ft (124.6 m) high above the deepest foundation level and 4756 ft long. The full reservoir level is +590.0 ft above the mean sea level (M.S.L.). The most unique feature of this dam is the adoption of Stone masonry for its construction, deviating from the traditional concrete. The use of stone had resulted in a large saving and had created huge employment potential for a large labour force. Hence, Nagarjuna Sagar Project ranks first in the man-power utilisation among the modern gigantic projects of its own kind in the world. It was designed and executed entirely by Indian engineers. On either side of the masonry dam, earth dams have been constructed for a length of about two miles, the maximum height being 85 ft. The spillway crest has been installed with 26 Radial gates each of size 45′×44′. Other component-works of this mighty dam include 8 penstock pipes on the left side, three Power sluices and 9 irrigation sluices on the right side, two chute sluices and a diversion-cum-irrigation tunnel. Two canals-off-take on either side of the dam for irrigation. The expenditure on the project was of the order of Rs. 80 crores. The crest level of the dam is +605 ft and the crest level of spillway is +546 ft. Various details of the dam are given below:

(1) **Location.** Lat. = 16°34′ North, Long. = 71°19′ East, $1\frac{1}{2}$ mile downstream of Nandikonda village Miryalaguda Taluk, Nalgonda District, 90 miles from Hyderabad.

(2) **States Covered.** ANDHRA PRADESH

(3) **Hydrology**

(*a*) Water-shed area at dam site	... 83,087 sq. miles
(*b*) Maximuml flood discharge (observed)	... 11.7 lakh cumecs.

(4) **Reservoir**

(*a*)	Full Reservoir level	... +590′.0
(*b*)	Maximum water level	... +594′.0
(*c*)	Gross storage capacity	... 9.37 M.a.ft
(*d*)	Net (live) storage capacity	... 5.51 M.a.ft
(*e*)	No. of villages submerged	... 57
(*f*)	Population displaced	... 4,824 families

(5) **Masonry dam**

(*a*)	Total length	... 4,756 ft
(*b*)	Spillway length	... 1545 ft
(*c*)	Non-overflow length including power dam	... 3,211 ft
(*d*)	Height of dam (maximum)	... 409 ft
(*e*)	Base width (maximum)	... 320 ft
(*f*)	Top width	... 28 ft

(*g*)	Top level	... + 605′.0
(*h*)	Top of crest in spillway	... + 546′.0
(*i*)	Chute sluices 2 No. 10′ × 25′ provided in blocks 25 and 51 with sill level elevation	... + 450′.0
(*j*)	8 No. of 16′ diameter penstocks provided one in each in blocks 16 to 23 with central line in elevation	... + 405′.0
(*k*)	3 No. power sluices 15′ × 38′ provided (two in block 71 and one in block 72) with sill level at elevation	... + 479′.0
(*l*)	Radial Crest Gates 26 No. 45′ × 44′, with crest elevation	... + 546′.0
(*m*)	Thickness of spillway pipes	... + 15′
(*n*)	Nine No. sluices 10′ × 15′ are provided for right canal head regulator with sill at elevation	... + 489′.0

(6) **Quantities of Work**

(*a*)	Excavation for foundations	... + 42.07 M. cft
(*b*)	Volume of masonry and concrete	... 198 M. cft
(*c*)	Quantity of cement	... About 11 lakh tons
(*d*)	Quantity of steel	... About 60,000 tons

(7) **Earth Dams**

(*a*)	Length of Left Earth Dam	... 8,400 ft
(*b*)	Length of Right Earth Dam	... 2,800 ft
		11,200 ft
(*c*)	Maximum height above foundation level	... 85 ft
(*d*)	Top width	... 30 ft
(*e*)	Top level	... 610′
(*f*)	Excavation of foundation	... 9.2 M. cft
(*g*)	Earthwork for embankment	... 88 M. cft

(8) **Irrigation-cum-Diversion Tunnel (horse-shoe in Section)**

(*a*)	Length	... 2,590 ft
(*b*)	Diameter	... 27 ft

(9) **Power Plant**

(*a*)	Left side 8 No. of penstocks 16 ft. diameter to develop	... 3 lakh kW
(*b*)	Right side 3 power sluices of size 15′ × 38′ to develop	... 0.6 lakh kW

(10) **Man-Power** ... 50,000

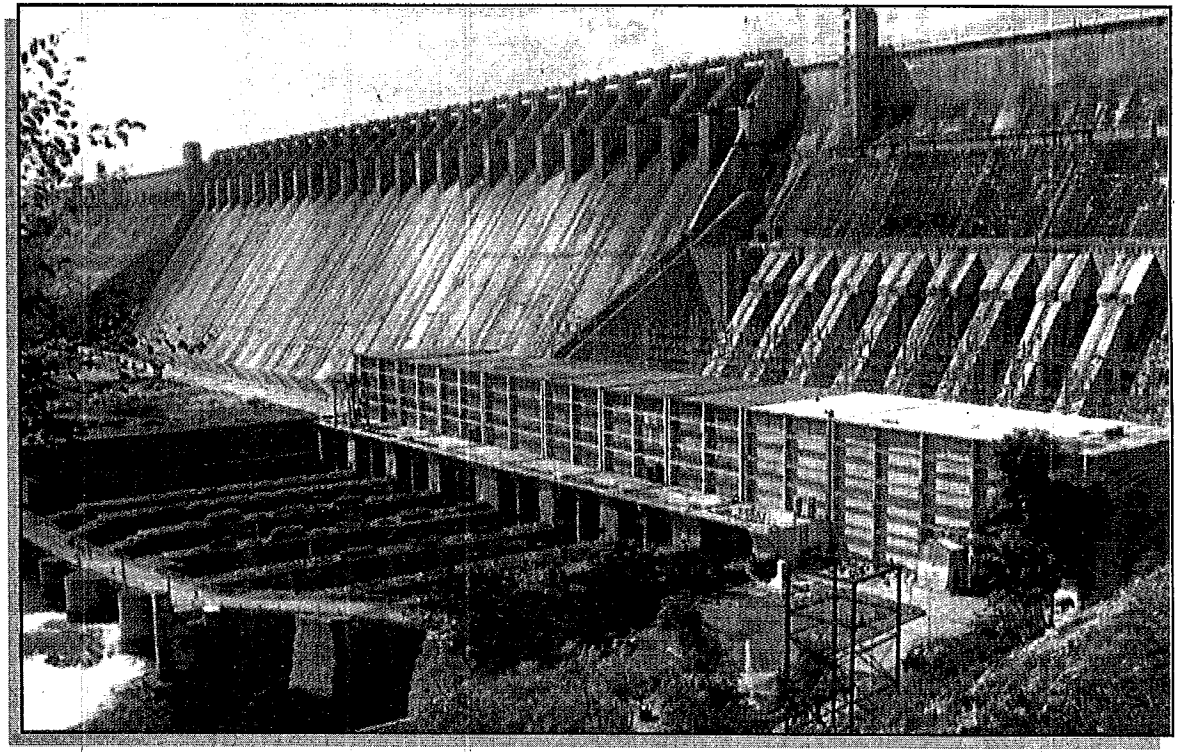


Fig. 17.5. Photoview of Nagarjuna Sagar Dam.

Organisation. In the earlier years of construction, a Control Board was the orverall incharge of the Nagarjun Sagar Project including technical, financial and administration aspects. The project was then located in the Andhra Pradesh and the Hyderabad States. After the reorganisation of States, the project came entirely in Andhra Pradesh State. The State Government assumed its full responsibility w.e.f. 1.8.1959. The Control Board became an advisory Body to the State Government, whose advice is accepted as a matter of convention.

The programme of construction of dam and canals was so adjusted that partial benefits started accruing even from 1967. The reservoir was able to deliver water to first crop of Krishna delta in time for raising substantial acreage in 2nd crop. Letting out waters in the two canals was inaugurated, by Smt. Indira Gandhi, the then Prime Minister of India, on 4.8.1967.

Rehabilitation. The man-made lake of Nagarjuna Sagar fully submerged lands of 52 villages and partially submerged that of 5 villages.

About 4,900 families were displaced and rehabilitated in 24 Rehabilitation centres. Good facilities, liberal compensation and other amenities were provided to those who had to be uprouted.

Nagarjunakonda Excavations. As the Nagarjuna Sagar reservoir was to completely submerge the famous relics of Nagarjuna Konda, which was the seat of Ikshwaku Kings and one of the principal centres of Mahayana System of Buddhism, the whole area was excavated by the Archeological Department of the Governmernt of India. The more important of the relics were located in a museum constructed for the purpose, on the top of an adjoining hill. Practically, all the relics had been unearthed and shifted to the museum. Expenditure to the extent of 12 lakh rupees was debited to the Project and the balance was met by the Archeological department.

PROBLEMS

1. (*a*) What is meant by a "dam and a reservoir" ? What are the different meterials that are commonly used for dam construction and what are their comparative advantages and disadvantages ?

(*b*) Discuss the geological and tropological features which affect the selection of the type of dam.

2. (*a*) What are 'arch' and 'buttress' dams ? Illustrate with sketches and mention site conditions favourable for construction of such dams.

(*b*) Discuss the factors which are considered in the selection of the site for a proposed dam. It is assumed that the type of the dam has already been selected for the project.

3. (*a*) What useful purpose is served by a dam ? What are the illeffects of dam construction ?

(*b*) How do you classify dams according to :

(*i*) their use ;

(*ii*) their hydraulic designs ;

(*iii*) their materials of construction.

(*c*) Discuss the various factors which govern the selection of a particular type of dam for a particular project.

4. "Dams are the sources of sorrow and grief". Debate the above statement giving points in favour as well as against it.

5. Narrate briefly the story of construction of any major dam of India.

18

Reservoirs and Planning for Dam Reservoirs

18.1. Definition and Types

When a barrier is constructed across some river in the form of a dam, water gets stored on the upstream side of the barrier, forming a pool of water, generally called a *dam reservoir* or an *impounding reservoir* or *a river reservoir.*

The quality of water stored in such a reservoir is not much different from that of a natural lake. The water so stored in a given reservoir during rainy season can be easily used almost throughout the year, till the time of arrival of the next rainy season, to refill the emptying reservoir again.

Depending upon the purpose served by a given reservoir, the reservoirs may be broadly divided into the following three types.

(1) *Storage or Conservation reservoirs* ;

(2) *Flood Control reservoirs* ; and

(3) *Multipurpose reservoirs.*

The fourth type of a reservoir is a simple storage tank constructed within a city water supply system, and is called a *Distribution reservoir* ; and such a reservoir is evidently not a river reservoir, but is a simple storage tank.

18.1.1. Storage or Conservation Reservoirs. A city water supply, irrigation water supply, or a hydroelectric project drawing water directly from a river or a stream may fail to satisfy the consumers demands during extremely low flows; while during high flows, it may become difficult to carry out their operations due to devastating floods. *A storage or a conservation reservoir can retain such excess supplies during periods of peak flows, and can release them gradually during low flows as and when the need arises.*

Incidentally, in addition to conserving water for later use, the storage of flood waters may also reduce flood damage below the reservoir. Hence, a reservoir can be used for controlling floods either solely or in addition to other purposes. In the former case, it is known as a 'Flood Control Reservoir' or a 'Single Purpose Flood Control Reservoir'; and in the latter case, it is called a 'Multipurpose Reservoir'.

18.1.2. Flood Control Reservoirs. A flood control reservoir, generally called a flood-mitigation reservoir, stores a portion of the flood flows in such a way as to minimise the flood peaks at the areas to be protected downstream. To accomplish this, the entire inflow entering the reservoir is discharged till the outflow reaches the safe capacity of the channel downstream. The inflow in excess of this rate is stored in the

reservoir, which is then gradually released, so as to recover the storage capacity for the next flood.

The flood peaks at the downstream of the reservoir are thus reduced by an amount *AB*, as shown in Fig. 18.1. A flood control reservoir differs from a conservation reservoir only in its need for a large sluiceway capacity to permit rapid drawdown before or after a flood.

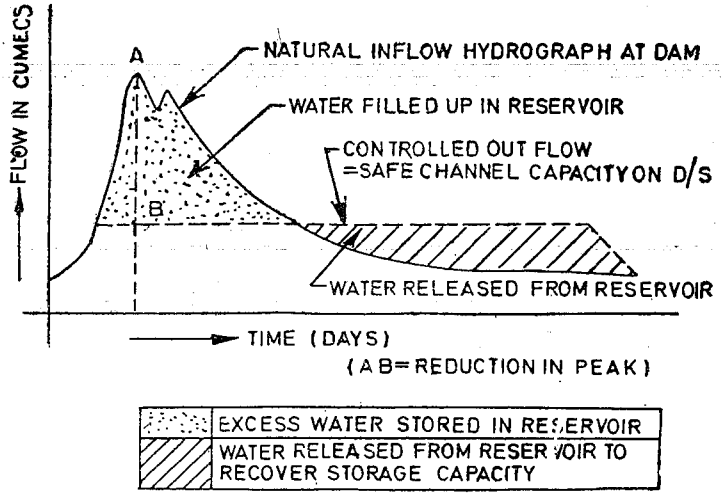


Fig. 18.1.

Types of flood control reservoirs. There are two basic types of flood-mitigation reservoirs; *i.e.*

(*i*) *Storage reservoirs or Detention basins* ; and

(*ii*) *Retarding basins or Retarding reservoirs.*

A reservoir having gates and valves installation at its spillway and at its sluice outlets is known as a storage reservoir; while on the other hand, a reservoir with uncontrolled and ungated outlets is known as a **retarding basin.**

Functioning and Advantages of a Retarding Basin

A retarding basin is usually provided with an uncontrolled spillway and an uncontrolled orifice type sluiceways. The automatic regulation of outflow, depending upon the availability of water, takes place from such a reservoir. The maximum discharging capacity of such a reservoir should be equal to the maximum safe carrying capacity of the channel downstream. As floods occur, the reservoir gets filled, and discharges through sluiceways. As the reservoir elevation increases, the outflow discharge increases. The water level goes on rising until the flood has subsided, and the inflow becomes equal to or less than the outflow. After this, the water gets automatically withdrawn from the reservoir until the stored water is completely discharged. The advantages of a retarding basin over a gate controlled detention basin are:

(*i*) Cost of the gate installation is saved.

(*ii*) There are no gates and hence, the possibility of human error and negligence in their operation is eliminated.

(*iii*) Since such a reservoir is not always filled, much of the land below the maximum reservoir level will be submerged only temporarily and occasionally, and can be successfully used for agriculture, although no permanent habitation can be allowed on this land.

Functioning and Advantages of a Storage Reservoir

A storage reservoir with gated spillway and gated sluiceways, provides more flexibility of operation, and thus gives us better control and increased usefulness of the reservoir. Storage reservoirs are, therefore, preferred on large rivers, which require better control; while retarding basins are preferred on small rivers. In storage reservoirs, the flood crest downstream, can be better controlled and regulated properly, so as not to cause their coincidence. This is the biggest advantage of such a reservoir and outweighs its disadvantages of being costly and involving risk of human error in installation and operation of gates.

18.1.3. Multipurpose Reservoirs. A reservoir planned and constructed to serve not only one purpose but various purposes together is called a multipurpose reservoir. Reservoir, designed for one purpose, incidentally serving other purposes, shall not be called a multipurpose reservoir, but will be called so, only if designed to serve those purposes also in addition to its main purpose. Hence, a reservoir designed to protect the downstream areas from floods and also to conserve water for water supply, irrigation, industrial needs, hydroelectric purposes, etc. shall be called a multipurpose reservoir. Bhakra dam and Nagarjun Sagar dam are the important multipupose projects of India.

18.1.4. Distribution Reservoirs. A distribution reservoir is a small storage reservoir constructed within a city water supply system. Such a reservoir can be filled by pumping water at a certain rate and can be used to supply water even at rates higher than the inflow rate during periods of maximum demands (called critical periods of demand). Such reservoirs are, therefore, helpful in permitting the pumps or the water treatment plants to work at a uniform rate, and they store water during the hours of no demand or less demand, and supply water from their 'storage' during the critical periods of maximum demand.

In this chapter, we shall however, confine ourselves to the river reservoirs only.

18.2. Capacity-Elevation and Area-Elevation Curves of a Reservoir Site

Whatever be the size or use of a reservoir, the main function of a reservoir is to store water and thus to stabilize the flow of water. Therefore, the most important physical characteristic of a reservoir is nothing but its *storage capacity*. The capacity of reservoirs on dam sites, is determined from the contour maps of the area. A topographic survey of the dam site is carried out, and a contour map such as shown in Fig. 18.2 is drawn. The area enclosed within each contour can be measured with a planimeter.

In fact, the general practice adopted for capacity computations is to actually survey the site contours only at vertical distances of 5 m or so. The areas of the intervening contours at smaller intervals of say 0.5 m or so, are then interpolated by taking the square root of the surveyed contours, and to assume that the square root of the interpolated ones, vary in exact proportion to their vertical distance apart. For example, suppose the area of the reservoir at 200 m contour is A_1 hectares, and that at 205 m contour is A_2 hectares; then

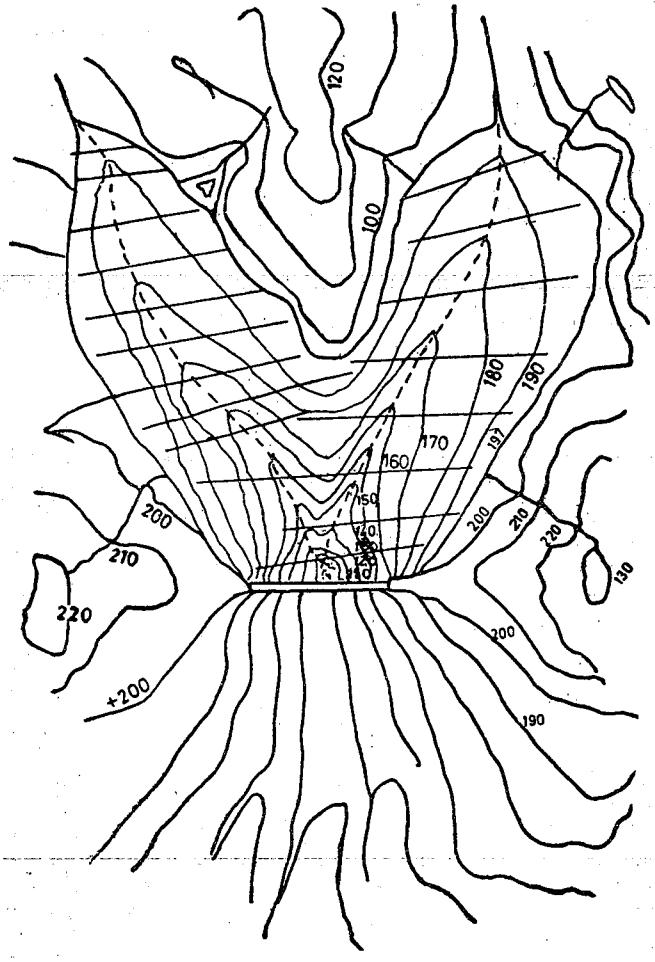


Fig. 18.2. A typical contour map of a dam site.

The area at 200.5 m contour is

$$= \left[\sqrt{A_1} + \frac{200.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{0.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad \text{...[18.1 (}a\text{)]}$$

Similarly, the area at 201.0 m contour

$$= \left[\sqrt{A_1} + \frac{201.0 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{1.0}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad \text{...[18.1 (}b\text{)]}$$

Similarly, the area at 204.5 m contour

$$= \left[\sqrt{A_1} + \frac{204.5 - 200}{205 - 200}(\sqrt{A_2} - \sqrt{A_1})\right]^2$$

$$= \left[\sqrt{A_1} + \frac{4.5}{5.0}(\sqrt{A_2} - \sqrt{A_1})\right]^2 \quad ...[18.1\ (c)]$$

In this way, the areas can be computed at sufficiently low contour intervals (0.5 m), which can be used to determine the incremental volumes (ΔS) stored between two successive contours, by using the simple average method, *i.e.* by multiplying the average of the two areas at the two elevations, by the elevation difference (Δh). The summation of these incremental volumes below any elevation, is the storage volume below that level.

Instead of using the simple average formula, *i.e.*

$$\Delta S = \frac{a_1 + a_2}{2}(\Delta h), \text{ sometimes the formula}$$

$$\Delta S = \frac{\Delta h}{3}\left[a_1 + a_2 + \sqrt{a_1 a_2}\right] \text{ can also be used, where}$$

a_1, a_2, ...represent the areas at 0.5 m contour interval.

Prismoidal formula can also be used preferably where three consecutive sections at equal height are taken. According to this,

$$\Delta S = \text{Storage} = \frac{\Delta h}{6}\left[A_1 + 4A_2 + A_3\right] \quad ...(18.2)$$

where A_1, A_2 and A_3 are the areas of succeeding contours, and Δh is the vertical distance between two alternate contours.

It thus becomes evident that the areas at different elevations (contours), as well as the storage at different elevations, can be mathematically worked out, and both plotted on a graph paper, to obtain **Area-Elevation curve, and Storage-Elevation curve** (*Capacity-Elevation curve*), respectively, as shown in Fig. 18.3.

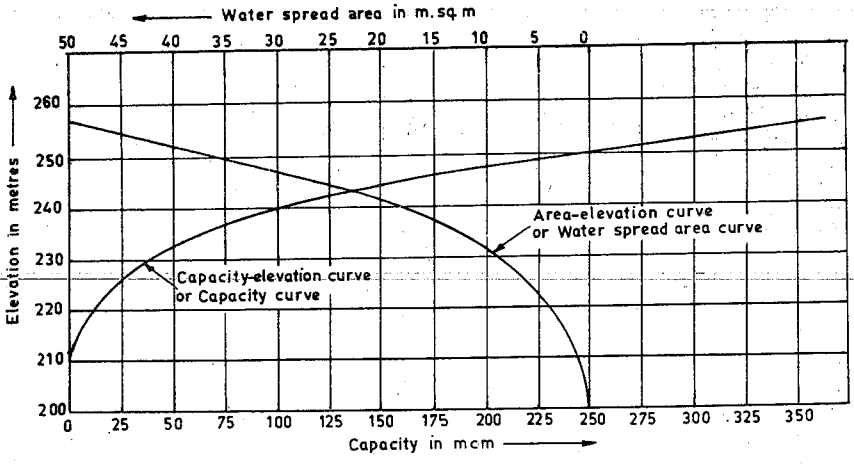


Fig. 18.3. Area elevation curve and Capacity elevation curve.

Another method for computing capacity may be the use of integration technique. Infact, the area-elevation curve, when integrated, will yield nothing but the capacity-elevation curve. Hence the surveyed areas at large-intervals, may be plotted on a simple graph paper, and a smooth curve, *i.e.* area-elevation curve, is first of all drawn. The equation of this curve is now obtained by statistical methods, which can be integrated to obtain the equation of the capacity-elevation curve, as follows:

The equation of the area-elevation curve, will generally be of the form:

$$A = \alpha + \beta \,.\, h + \gamma \,.\, h^2 + \ldots \eta \,.\, h^{n-1} \qquad \ldots(18.3)$$

where A represents the area at any elevation h; and
$\alpha, \beta, \gamma \ldots \eta$ are all constants.

This equation can be determined and then integrated to obtain the storage (capacity), as explained below:

Let y be the height of the water surface in the reservoir above any assumed datum, over which the storage/capacity is to be worked out.

Let A_y represents symbolically the area of the contour at this height. Then, assume that the *equation of area-elevation curve* is given by :

$$A_y = \alpha + \beta \,.\, y + \gamma \,.\, y^2 + \ldots \eta \,.\, y^{n-1} \qquad \ldots(18.3\ a)$$

where $\alpha, \beta, \gamma, \ldots, \eta$ are all constants.

From the actual survey, or from the points falling on the area-elevation curve, the area of any required number of contours (n) are known.

Let them be A_0, A_1, A_2.....corresponding to the known level values, *i.e.*, heights $0, y_1, y_2, \ldots$ above the datum.

Substituting these values in equation (18.3 *a*), we get

$$A_0 = \alpha$$
$$A_1 = \alpha + \beta \,.\, y_1 + \gamma \,.\, y_1^2 + \ldots \eta \,.\, y_1^{n-1}$$
$$A_2 = \alpha + \beta \,.\, y_2 + \gamma \,.\, y_2^2 + \ldots \eta \,.\, y_2^{n-1}$$
$$A_3 = \alpha + \beta \,.\, y_3 + \gamma \,.\, y_3^2 + \ldots \eta \,.\, y_3^{n-1}$$

Thus, we get n simultaneous equations to determine n number of constants $(\alpha, \beta, \gamma, \ldots, \eta)$ Hence, the equation of the area-elevation curve, *i.e.* $A_y = \alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1}$ becomes defined, with $\alpha, \beta, \gamma, \ldots \eta$ all known.

This equation can now be integrated between the limits 0 to y,

$$\int_{y=0}^{y=y} A_y \cdot dy = \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

or S_y = storage (capacity) between 0 to y

$$= \int_{y=0}^{y=y} (\alpha + \beta.y + \gamma.y^2 + \ldots \eta.y^{n-1})\, dy$$

$$= \left[\alpha.y + \beta.\frac{y^2}{2} + \gamma.\frac{y^3}{3} + \ldots \eta.\frac{y^n}{n} \right] + K$$

where K is a constant, which obviously is the reservoir capacity at datum.

The use of this method can be best understood by solving a numerical example.

Example 18.1. *A contour survey of a reservoir site gives the following data:*

Contour value	*Area*
At 200 m contour	*6.0 hectares*
At 210 m contour	*18.1 hectares*
At 220 m contour	*34.0 hectares*

The capacity of the reservoir upto 200 m elevation is found to be 14.1 ha. m. Determine the general equation for the area-elevation curve and capacity-elevation curve. Also determine the reservoir capacity at RL 225 m.

Solution. Use equation (18.3 *a*), wherein y is the height above *RL* 200 m, as

$$A_y = \alpha + \beta \,.\, y + \gamma \,.\, y^2$$

Now substituting the given values, we have

$$A_0 = 6.0 \text{ hectares}$$

$$\therefore \quad 6.0 = \alpha + \beta\,(0) + \gamma\,(0) = \alpha$$

or $$\alpha = \mathbf{6.0} \qquad ...(i)$$

Also $A_1 = 18.1$ hectares, at $y_1 = 10$ m (given)

$$\therefore \quad 18.1 = \alpha + \beta \,.\, (10) + \gamma \,.\, (10)^2$$

or $$18.1 = 6.0 + 10\beta + 100\gamma$$

or $$12.1 = 10\beta + 100\gamma$$

or $$\beta + 10\gamma = 1.21 \qquad ...(ii)$$

Also $A_2 = 34.0$ hectares at $y_2 = 20$ m (given)

$$\therefore \quad 34.0 = \alpha + \beta\,(20) + \gamma\,(20)^2 = 6.0 + 20\beta + 400\gamma$$

or $$20\beta + 400\gamma = 28.0$$

$$\beta + 20\gamma = 1.4 \qquad ...(iii)$$

Solving (*ii*) and (*iii*), we get

$$\beta + 10\gamma = 1.21$$

$$\beta + 20\gamma = 1.40$$

or $$10\gamma = 0.19 \quad \text{or} \quad \gamma = \mathbf{0.019}$$

$$\therefore \quad \beta + 0.19 = 1.21 \quad \text{or} \quad \beta = \mathbf{1.02}$$

Hence, the equation of area-elevation curve is given by

$$A_y = 6.0 + 1.02y + 0.019y^2 \quad \textbf{Ans.}$$

Integrating this equation, we get

$$S_y = 6.0y + 1.02\frac{y^2}{2} + 0.019\frac{y^3}{3} + K$$

The constant K is obviously the reservoir capacity (storage) upto *RL* 200.0 m, which is given to be 14.1 ha. m.

$$\therefore \quad S_y = 6.0y + 0.51y^2 + 0.0063y^3 + 14.1$$

Hence, the capacity-elevation curve is given by

$$S_y = 0.0063y^3 + 0.51y^2 + 6.0y + 14.1 \quad \textbf{Ans}$$

To determine the capacity at *RL* 225 m, we have to substitute $y = 225 - 200 = 25$ m in the above eqn. to obtain the requisite capacity in ha.m., *i.e.*

Required capacity at *RL* 225 m

$$= 0.0063\,(25)^3 + 0.51\,(25)^2 + 6.0 \times 25 + 14.1$$
$$= 98.96 + 318.75 + 150.0 + 14.1 = \mathbf{581.81\,ha.m.} \quad \textbf{Ans.}$$

Conclusions. It can thus be seen that the capacity of a reservoir by this method can be determined by surveying only a few contours. It is also found that the method does not give more than 3% error, when it is cross-checked with the capacity worked out by surveying large number of contours. This error is not considered much, in the light of the fact that the areas of contours are themselves not very precise figures.

Sometimes, storage capacity may be expressed as a single term function of y, such as

$$S_y = K.y^n \qquad \text{where } K \text{ and } n \text{ are constants.}$$

Infact, in practical life, no one bothers about the equations, and only curves, as shown in Fig. 18.3 are drawn. The required capacity at any elevation is read out, from such a curve.

It may also be pointed out here that the best capacity curve for a reservoir is the one in which the rise to the straight line is the quickest. It results from a cup-shaped catchment, with gentle longitudinal slope. Fig. 18.4 shows the good and bad capacity curves.

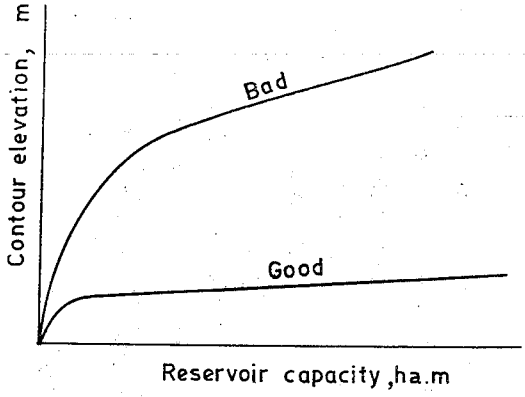


Fig. 18.4. Good and Bad capacity curves for a reservoir

18.3. Storage Zones of a Reservoir

These zones are defined w.r. to Fig. 18.5.

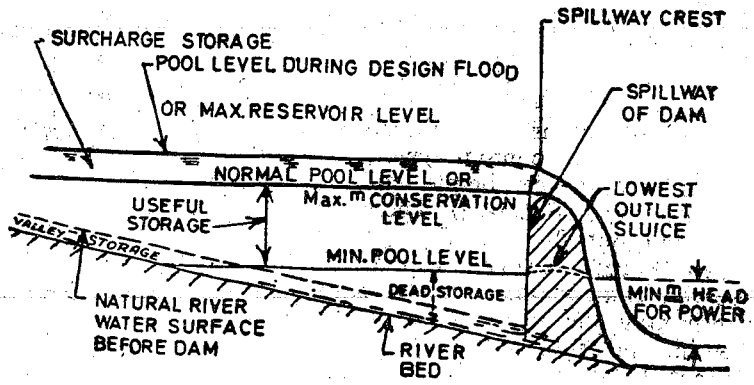


Fig. 18.5. Storage zones of a reservoir.

18.3.1. Normal Pool Level or Maximum Conservation Level. It is the maximum elevation to which the reservoir water surface will rise during normal operating conditions. (See Fig. 18.5). It is equivalent to the elevation of the spillway crest or the top of the spillway gates, for most of the cases.

18.3.2. Minimum Pool Level. The lowest water surface elevation, which has to be kept under normal operating conditions in a reservoir, is called the minimum pool level (See Fig. 18.5). This level may be fixed by the elevation of the lowest outlet in the dam or may be guided by the minimum head required for efficient functioning of turbines.

18.3.3. Useful and Dead Storage. *The volume of water stored in a reservoir between the minimum pool and normal pool levels is called the useful storage.* Water stored in the reservoir below the minimum pool level is known as the *Dead Storage,* and it is not of much use in the operation of the reservoirs. The useful storage may be subdivided into conservation storage and flood-mitigation storage, in a multipurpose reservoir.

18.3.4. Maximum Pool Level or Full Reservoir Level. During high floods, water is discharged over the spillway, but will cause the water level to rise in the reservoir above the normal pool level. The maximum level to which the water rises during the worst design flood is known as the *maximum pool level.*

18.3.5. Surcharge Storage. The volume of water stored between the normal pool level and the maximum pool level is called *surcharge storage.* Surcharge storage is an uncontrolled storage, in the sense that it exists only till the flood is in progress and cannot be retained for later use.

18.3.6. Bank Storage. When the reservoir is filled up, certain amount of water seeps into the permeable reservoir banks. This water comes out as soon as the reservoir gets depleted. This volume of water is known as the *bank storage,* and may amount to several percent of the reservoir volume depending upon the geological formations. The bank storage effectively increases the capacity of the reservoir above that indicated by the elevation capacity curve of the reservoir.

18.3.7. Valley Storage. Even before a dam is constructed, certain variable amount of water is stored in the stream channel, called *valley storage.* After the reservoir is formed, the storage increases, and the actual net increase in the storage is equal to the storage capacity of the reservoir minus the natural valley storage. The valley storage thus reduces the effective storage capacity of a reservoir. It is not of much importance in conservation reservoirs, but the available storage for flood mitigation is reduced, as given by the following relation:

Effective storage for flood mitigation
= Useful Storage + Surcharge Storage – Valley Storage corresponding to the rate of inflow in the reservoir.

DESIGNING RESERVOIR CAPACITY

18.4. Catchment Yield and Reservoir Yield

Long range runoff from a catchment is known as the yield of the catchment. Generally, a period of one year is considered for determining the yield value. *The total yearly runoff, expressed as the volume of water entering/passing the outlet point of the catchment, is thus known as the catchment yield, and is expressed in M m^3 or M.ha.m.*

The annual yield of the catchment upto the site of a reservoir, located at the given point along a river, will thus indicate the quantum of water that will annually enter the reservoir, and will thus help in designing the capacity of the reservoir. This will also help to fix the outflows from the reservoir, since the outflows are dependent upon the inflows and the reservoir losses.

The amount of water that can be drawn from a reservoir, in any specified time interval, called the *reservoir yield,* naturally depends upon the inflow into the reservoir and the reservoir losses, consisting of reservoir leakage and reservoir evaporation.

The annual inflow to the reservoir, i.e. the *catchment yield,* is represented by the *mass curve of inflow;* whereas, the outflow from the reservoir, called the *reservoir yield,* is represented by the *mass demand line* or *the mass curve of outflow.* Both these curves decide the reservoir capacity, provided the reservoir losses are ignored or separately accounted.

The inflows to the reservoir are however, quite susceptible of variation in different years, and may therefore vary throughout the prospective life of the reservoir. *The past available data of rainfall or runoff in the catchment is therefore used to work out the optimum value of the catchment yield.* Say for example, in the past available records of say 35 years, the minimum yield from the catchment in the worst rainfall year may be as low as say 100 M.ha.m; whereas, the maximum yield in the best rainfall year may be as high as say 200 M.ha.m. The question which then arises would be as to whether the reservoir capacity should correspond to 100 M. ha.m yield or 200 M. ha.m yield. If the reservoir capacity is provided corresponding to 100 M. ha.m yield, then eventually the reservoir will be filled up every year with a dependability of 100%; but if the capacity is provided corresponding to 200 M.ha.m yield, then eventually the reservoir will be filled up only in the best rainfall year (*i.e.* once in 35 years) with a dependability of about $\frac{1}{35} \times 100 \approx 3\%$.

In order to obtain a sweet agreement, a via media is generally adopted and an intermediate *dependability percentage value* (p), such as 50% to 75%,* may be used to compute the **dependable yield** or **the design yield.** The yield which corresponds to the worst or the most critical year on record is however, called the **firm yield** or **the safe yield.** Water available in excess of the firm yield during years of higher inflows, is designated as **the secondary yield.** Hydropower may be developed from such secondary water, and sold to the industries *'on and when available basis'.* The power commitments to domestic consumers must, however, be based on the firm basis, and should not exceed the power which can be produced with the firm yield, unless thermal power is also available to support the hydroelectric power.

The arithmetic average of the firm yield and the secondary yield is called the **average yield.**

18.4.1. Computing the Design or the Dependable Catchment Yield. The dependable yield, corresponding to a given dependability percentage p, is determined from the past available data of the last 35 years or so. The yearly rainfall data of the reservoir catchment is generally used for this purpose, since such long runoff data is rarely available . The rainfall data of the past years is therefore used to work out the dependable rainfall value corresponding to the given dependability percentage p. This dependable rainfall value is then converted into the dependable runoff value by using the available empirical formulas connecting the yearly rainfall with the yearly runoff.

It is, however, an adopted practice in Irrigation Departments to plan the reservoir project by computing the dependable yield from the rainfall data, but to start river gauging as soon as the site for the reservoir is decided, and then correlate the rainfall-runoff observations to verify the correctness of the assumed empirical relation between the rainfall and the runoff. Sometimes, on the basis of such observations, the initially assumed yield value may have to be revised.

* This percentage will depend upon the risk which can be absorbed for the proposed use of water. Say for example, city water supply projects can absorb lesser risk as compared to the irrigation projects, and hence should consider higher dependability percentage value.

The procedure which is adopted to compute the dependable rainfall value for a given dependability percentage p is explained below, and has been further used in solving example 18.2.

(*i*) The available rainfall data of the past N years is first of all arranged in the descending order of magnitude.

(*ii*) The order number m, given by the equation

$$m = N \cdot \frac{p}{100} \qquad ...(18.4)$$

is then computed, and rainfall value corresponding to this order number in the tabulated data will represent the required dependable rainfall value.

(*iii*) If the computed value of m is a fraction, then the arithmetic mean of the rainfall values corresponding to whole number m values above and below this fraction value is taken as the **dependable rainfall** value.

This method of computing the rainfall value of the given dependability (such as 50%, 75%, etc.) will become more clear when we solve example 18.2. The rainfall value of given dependability can finally be converted into the dependable yield, by using empirical methods, discussed in article 18.4.1.1.

The dependable yield of a given dependability (p per cent) can also be determined from the stream gauging, by computing the annual yields, from the observed discharges, and arranging the annual yield values in descending order to determine order numbers (instead of rainfall) as exhibited in solved example 18.3.

Example 18.2. *The yearly rainfall data for the catchment of a proposed reservoir site for 35 years is given in Table 18.1. Compute from this data, the values of dependable rainfalls for 60% and 75% dependability percentage.*

Table 18.1

Year	*Rainfall in cm*	*Year*	*Rainfall in cm*
1956	98	1974	88
1957	100	1975	94
1958	101	1976	107
1959	99	1977	110
1960	85	1978	208
1961	112	1979	114
1962	116	1980	104
1963	78	1981	120
1964	160	1982	108
1965	66	1983	102
1966	184	1984	80
1967	90	1985	109
1968	76	1986	122
1969	118	1987	115
1970	86	1988	140
1971	92	1989	138
1972	96	1990	60
1973	93		

Solution. The given data of table 18.1 is arranged in descending order, mentioning serial number (order number m) in front of each, as shown in Table 18.2.

Table 18.2

S.No. i.e order number (m)	*Rainfall in desecending order in cm*	*S. No. i.e. order number (m)*	*Rainfall in descending order in cm*
1	208	19	101
2	184	20	100
3	160	21	99
4	140	22	98
5	138	23	96
6	122	24	94
7	120	25	93
8	118	26	92
9	116	27	90
10	115	28	88
11	114	29	86
12	112	30	85
13	110	31	80
14	109	32	78
15	108	33	76
16	107	34	66
17	104	35	60
18	102		

Now, using equation (18.4), we compute the order number (m) for the given dependability percentage $p = 60\%$, as:

$$m = N \cdot \frac{p}{100} = 35 \times \frac{60}{100} = 21$$

The rainfall value tabulated above in table 18.2 at order no.21 is 99 cm; and hence the required dependable rainfall = 99cm. $\therefore P_{60\%} = \mathbf{99\ cm}$ Ans.

Similarly, for dependability $p = 75\%$, we calculate the order no. (m)

$$= N \cdot \frac{p}{100} = 35 \times \frac{75}{100} = 26.25.$$

Since reqd. order no. is not an integer, the mean value of rainfall corresponding to order no. 26 and 27 will be taken as the value of p of 75% dependability.

$$\therefore \qquad P_{75\%} = \frac{92 + 90}{2} = 91 \text{ cm} \quad \textbf{Ans.}$$

Example 18.3. *The daily flows in a river for three consecutive years are given in the table by class interval alongwith the number of days the flow belonged to this class. What are the 50% and 75% dependable flows (annual and daily) for the river ?*

Table 18.3

		Year 1981	*Year 1982*	*Year 1983*
S. No.	*Daily mean discharge m^3/s (range)*	*No. of days the flow belonged to the given range (class interval)*	*No. of days the flow equalled the class range of col (2)*	*No. of days the flow equalled the range given in col (2)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
1	*100-90.1*	*0*	*6*	*10*
2	*90-80.1*	*16*	*19*	*16*
3	*80-70.1*	*27*	*25*	*38*
4	*70-60.1*	*21*	*60*	*67*
5	*60-50.1*	*43*	*51*	*58*
6	*50-40.1*	*59*	*38*	*38*
7	*40-30.1*	*64*	*29*	*70*
8	*30-20.1*	*22*	*48*	*29*
9	*20-10.1*	*59*	*63*	*26*
10	*10-negligible*	*54*	*26*	*13*

(Engg. Services, 2002)

Solution. With the given daily flows, we will calculate the annual yields for the given 3 years of records, as computed in Table 18.4. Since daily discharge figs. are given in range (interval), mid value of range (interval) can be assumed to be the flow discharge, which when multiplied by no. of days, will give yield during those days. Summation of yield values over 365 days will give annual yield, as shown in Table 18.4, which otherwise is self explanatory.

Table 18.4

			Year 1981		Year 1982		Year 1983	
S. No.	Daily mean discharge (range) (m^3/s) in descending order	Mean value of daily discharge range	No. of days for which the flow equalled to that given in col (3)	Yield (Mm^3) = (3) × (4) × $\frac{86400}{10^6}$ [(3) × (4) × 0.0864]	No. of days for which flow equalled to that of col (3)	Yield (Mm^3) = 0.0864 × col (3) × col (6)	No. of days for which flow equaled to that of col (3)	Yield ($M\,m^3$) = 0.0864 × col (3) × col (8)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	100-90.1	95.05	0	0	6	49.25	10	82.08
2	90-80.1	85.05	16	117.50	19	139.54	16	117.50
3	80-70.1	75.05	27	174.96	25	162.00	38	246.24
4	70-60.1	65.05	21	117.94	60	336.96	67	376.27
5	60-50.1	55.05	43	204.34	51	242.35	58	275.62
6	50-40.1	45.05	59	229.39	38	147.77	38	147.74
7	40-30.1	35.05	64	193.54	29	87.70	70	211.68
8	30-20.1	25.05	22	47.52	48	103.68	29	62.64
9	20-10.1	15.05	59	74.34	63	81.65	26	33.70
10	10-Negligible	5.05	54	23.33	26	11.23	13	5.62
	Σ		365	1182.86	365	1361.98	365	1559.09

The river yield for 3 year is thus, calculated by summation of col (5), (7) & (9). Let these yields be arranged in descending order.

Year	Annual yield ($M\,m^3$)	Order No. (m)
1983	1559.09	1
1982	1361.98	2
1981	1182.86	3
		$N = 3$

Order No. (m) for p% dependability

$$= \frac{p}{100} \times N \qquad \text{...(Eqn. 18.4)}$$

(1) Order No for 75% dependability $= \frac{75}{100} \times 3 = 2.25$.

Since order No. of 2.25 is not an integer, the mean value of yield corresponding to S. NO. 2 and 3 will be taken as yield for 75% dependability, given as

$$= \left[\frac{1361.98 + 1182.86}{2}\right] = 1272.42\,M\,m^3$$

Hence, Annual flow with 75% dependability = **1272.42 M m³** **Ans.**

∴ Daily flow (discharge) with 75% dependability

$$= \frac{1272.42 \times 10^6}{365 \times 86400}\ m^3/s = \mathbf{40.34^3\ m^3/s} \quad \textbf{Ans.}$$

(*ii*) Order No. for 50% dependability $= \frac{p}{100} \times N = \frac{50}{100} \times 3 = 1.5$

Since order No. of 1.5 is not an integer, the mean value of yield corresponding to order No. 1 and 2 shall be taken as the yield of 50%. Dependability, given as :

$$= \frac{1559.09 + 1361.98}{2} = 1460.54\, M\, m^3$$

Hence, Annual flow with 50% dependability = **1460.54 M m^3** **Ans.**

∴ Daily flow (discharge) with 50% dependability

$$= \frac{1460.54 \times 10^6}{365 \times 86400}\, m^3/s = \mathbf{46.31\ m^3/s} \quad \textbf{Ans.}$$

18.4.1.1. Converting the dependable rainfall value into the dependable yield value. Certain empirical relations are available for converting the yearly rainfall value for the given catchment into the yearly runoff value expected from that catchment. Some of these formulas are:

1. *Binnie's percentages;*
2. *Strange's tables;*
3. *Barlow's tables;*
4. *Lacey's formula;*
5. *Inglis formula; and*
6. *Khosla's formula.*

These empirical relations are briefly discussed below:

(1) **Binnie's percentages.** The first effort ever made in India to connect the long range rainfall and the runoff (yield), was from Sir Alexander Binnie. He made observations *on two rivers in the central provinces,* and worked out certain percentages to connect the monthly rainfall with the monthly yield for the entire monsoon period from June to October. These percentages have been further adjusted by Mr. Garret, and are given in table 18.5. From the values given in table 18.5, *the total monsoon rainfall Vs percentage of rainfall that becomes runoff* are plotted in Fig. 18.6(*a*).

Table 18.5. Binnie's Monsoon Yield Percentages (as adjusted by Garret)

Monsoon Rainfall (P) in cm	*Monsoon Yield percentage*	*Monsoon Rainfall (P) in cm*	*Monsoon Yield percentage*
25	7.0	107.5	39.5
30	9.0	110.0	40.3
35	11.0	112.5	41.0
40	13.0	115.0	41.7
45	15.0	117.5	42.4
50	17.0	120.0	43.1
55	19.0	122.5	43.7
60	21.0	125.0	44.4
65	23.0	127.5	45.0
70	25.0	130.0	45.6
75	27.0	132.5	46.2
80	29.0	135.0	46.8
85	31.0	137.5	47.3
90	33.0	140.0	47.9
95	35.0	142.5	48.4
100	37.0	145.0	49.0
102.5	37.9	147.5	49.5
105.0	38.7	150.0	50.0

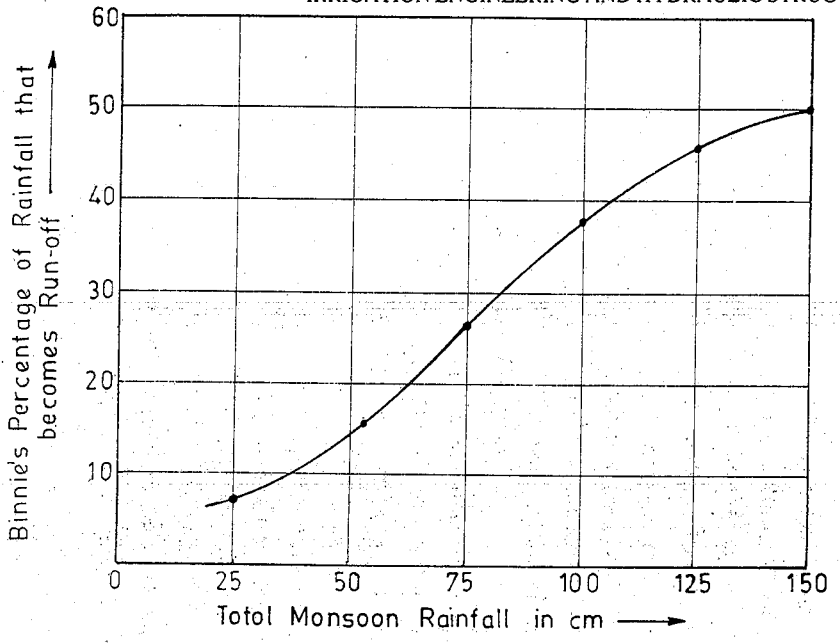


Fig. 18.6(*a*). Binnie's Monsoon Rainfall-runoff Curve.

(2) **Strange's percentages and tables.** Mr. W.L. Strange carried out investigations on catchments in Bombay Presidency, and worked out percentages for converting monsoon rainfall into monsoon yield. He even worked out such percentages for converting daily rainfalls into daily runoffs. It was an improvement over Binnie's tables, since he divided the catchments into three categories to account for the general characteristics of the catchments. The catchments prone to producing higher yields, such as those with more paved areas, etc. were categorised as *good catchments;* and those prone to producing low yields were termed as *bad catchments.* The intermediate types were called *average catchments.* Different runoff percentages were given for different types of catchments for different values of rainfalls, as shown in table 18.6(*a*). These values have also been drawn in the shape of curves, as shown in Fig. 18.6(*b*).

Table 18.6. Values of Strange's Run off Percentages

Monsoon Rainfall in cm	*Runoff Percentages for catchments designated as*			*Monsoon Rainfall in cm*	*Runoff Percentages for catchments designated as*		
	Good	*Average*	*Bad*		*Good*	*Average*	*Bad*
25.0	4.3	3.2	2.1	107.5	40.9	30.6	20.4
30.0	6.2	4.6	3.1	110.0	42.0	31.5	21.0
35.0	8.3	6.2	4.1	112.5	43.1	32.3	21.5
40.0	10.5	7.8	5.2	115.0	44.3	33.2	22.1
45.0	12.8	9.6	6.4	117.5	45.4	34.0	23.2
50.0	15.0	11.3	7.5	120.0	46.5	34.8	23.8
55.0	17.3	12.9	8.6	122.5	47.6	35.7	23.8
60.0	18.5	14.6	9.7	125.0	48.8	36.6	24.4
65.0	21.8	16.3	10.9	127.5	49.9	37.4	24.9
70.0	24.0	18.0	12.0	130.0	51.0	38.2	25.5
75.0	26.3	19.7	13.1	132.5	52.0	39.0	26.0
80.0	28.5	21.3	14.2	135.0	53.3	39.9	26.6
85.0	30.8	23.1	15.4	137.5	54.4	40.8	27.2
90.0	33.0	24.7	16.5	140.0	55.5	41.6	27.7
95.0	35.3	26.4	17.6	142.5	56.6	42.4	28.3
100.0	37.5	28.1	18.7	145.0	57.8	43.3	28.9
102.5	38.5	28.9	19.3	147.5	58.9	44.1	29.4
105.0	39.5	29.8	19.9	150.0	60.0	45.0	30.0

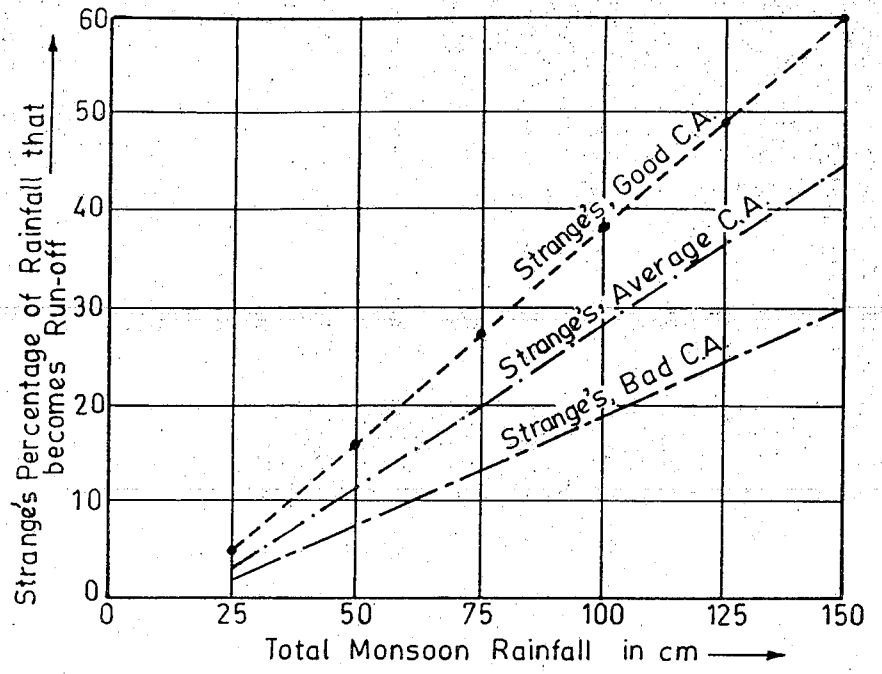


Fig. 18.6(*b*). Strange's monsoon Rainfall-Runoff curves.

(3) **Barlow's tables.** Mr. T.G. Barlow, the first Chief engineer of Hydro-Electric Survey of India, carried out extensive investigations on catchments mostly under 130 square km in U.P. State of India. On the basis of his investigations, he divided catchments into five classes and assigned different percentages of rainfall that become runoff (over long periods) for each class of catchment, as indicated in table 18.7.

Table 18.7. Values of Barlow's Runoff Percentages

S.No.	*Class of catchment*	*Description of the catchment*	*Runoff percentages (K)*
1.	A	Flat, cultivated, adsorbent soils	10
2.	B	Flat, partly cultivated stiff soils	15
3.	C	Average catchment	20
4.	D	Hills and plains with little cultivation	25
5.	E	Very hilly and steep catchment with little or no cultivation	33

The above values of runoff percentages are for the average type of monsoons, and are to be modified by the application of the following coefficients (Table 18.8) according to the nature of the season.

Table 18.8. Values of Barlow's coefficient to be multiplied with K values of table 18.7 to obtain True Runoff Percentages

S.No.	*Nature of season*	*Class of catchment*				
		A	*B*	*C*	*D*	*E*
1.	Light rain, no heavy down pour	0.7	0.8	0.8	0.8	0.8
2.	Average or varying rainfall, no continuous down pour	1.0	1.0	1.0	1.0	1.0
3.	Continuous down pour	1.5	1.5	1.6	1.7	1.8

(4) **Lacey's formula.** This formula connects the monsoon *rainfall (P) with the yield (Q)* by the equation.

$$\text{Yield } (Q) = \left[\frac{P}{1 + \frac{304.8\,m}{p.n}}\right] \qquad ...(18.5)$$

where m = a constant, called *monsoon duration factor,* the values of which are given in table 18.9.

n = a constant, called *catchment factor,* the values of which are given for different classes of catchments (as defined by Barlow) in table 18.10.

Table 18.9. Values of m to be used in Eq. (18.5)

S.No.	*Duration of Monsoon*	*Monsoon Duration factor m*
1.	Bad year	0.5
2.	Normal year	1.2
3.	Good year	1.5

Table 18.10. Values of *n* to be used in Eq. (18.5)

S.No.	*Class of catchment*	*Description of the catchment*	*Values of n*
1.	A	Flat cultivated, absorbent soils	0.25
2.	B	Flat, partly cultivated stiff soils	0.60
3.	C	Average catchments	1.00
4.	D	Hills and plains with little cultivation	1.70
5.	E	Very hilly and steep catchments with little or no cultivation	3.45

(5) **Inglis formula.** Inglis derived his formula for catchments of West Maharashtra State of India. He divided the areas as ghat areas (*Sahyadri ranges*) where rainfall is 200 cm or more; and non-ghat areas where rainfall is less than 200 cm. His formulas are:

(*a*) *For ghat areas, with rainfall (P) equalling or exceeding 200 cm:*

$$\text{Yield} = (0.85P - 30.48) \text{ cm} \qquad ...(18.6)$$

where P is the rainfall in cm.

(*b*) *For non-ghat areas with rainfall P less than 200 cm.*

$$\text{Yield} = \frac{P\,(P - 17.78)}{254} \text{ cm} \qquad ...(18.7)$$

where P is the rainfall in cm.

(6) **Khosla's formula.** This formula is based upon the recent research work conducted in this field, and is a very simple and useful formula. It can be easily applied to the entire country, without bothering for the region of its origin. This formula states that

$$\text{Yield } (Q) = P - 0.48\,T_m \qquad ...(18.8)$$

where Q = the yield in cm

P = the rainfall in cm

T_m = mean annual temperature of the area.

Example 18.4. *The design annual rainfall for the catchment of a proposed reservoir has been computed to be 99 cm. The catchment area has been estimated to have the mean annual temperature of 20°C. The catchment area contributing to the proposed reservoir is 1000 sq.km. Calculate the annual design catchment yield for this reservoir. Make use of Khosla's formula.*

Solution. Use Khosla's formula connecting the design rainfall (P) with the design yield (Q) by Eq. (18.8) as:

$$Q = P - 0.48\, T_m$$

where $P = 99$ cm (given)

$T_m = 20°C$ (given)

$$\therefore \quad Q = 99 - 0.48 \times 20 = 89.4 \text{ cm} = \mathbf{0.894\ m}$$

The total yield produced in m^3 from the given catchment of 1000 sq.km (*i.e.* $1000 \times 10^6\ m^2$)

$$= 0.894 \times 1000 \times 10^6\ m^3 = 894\ M.m^3 \quad \textbf{Ans.}$$

18.4.2. Use of Flow Duration Curves for Computing Dependable Flow

As we know, the stream flow varies widely over a **water year***. This variability of stream flow can be studied by plotting *flow duration curves* for the given stream. A **flow duration curve**, also called as **discharge-frequency curve**, is a curve plotted between *stream flow (Q)* and *percent of time the flow is equalled or exceeded (P_p)*, and is of the type shown in Fig. 18.7 (*a*), (*b*) and (*c*).

Such a curve can be plotted by first arranging the stream flow values (Q) in descending order using class intervals, if the number of individual values is very large. The data to be used can be of daily values, weekly values, or monthly values. If N data values are used, the plotting position of any discharge (or class value) Q is given, as :

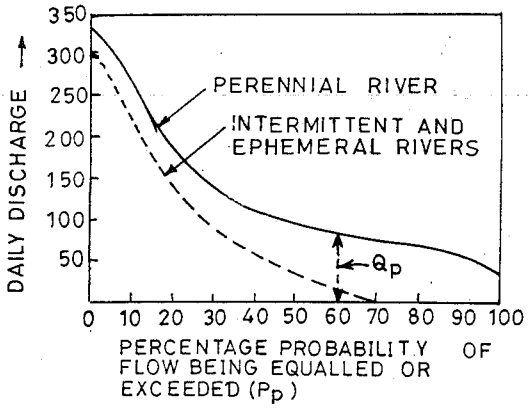


Fig. 18.7(*a*). Typical flow-duration curves on an ordinary paper.

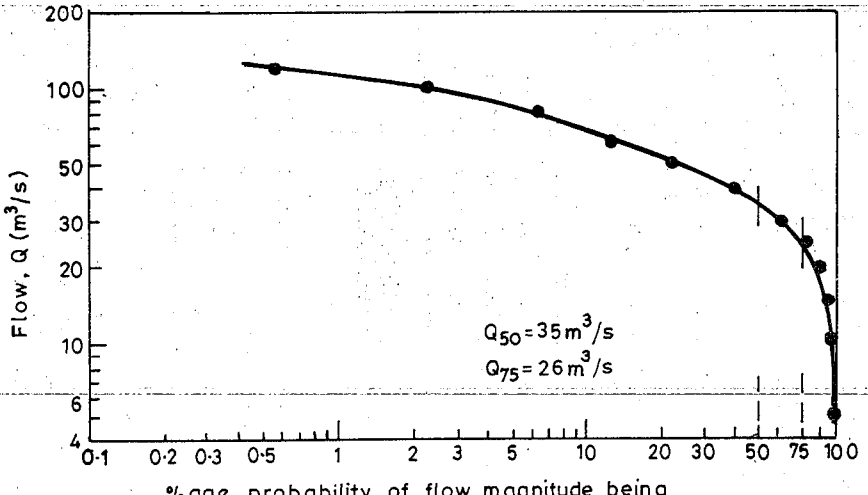


Fig. 18.7(*b*). A typical flow duration curve on a log-log paper.

* 1st June to 30st May.

$$P_p = \frac{m}{N+1} \times 100\%$$

where m = is the order No. of that discharge (or class value)

P_p = percentage probability of the flow magnitude being equalled or exceeded.

The ordinate Q at any percentage probability p (such as 60%), *i.e.* Q_p, will represent the flow magnitude of the river that will be available for 60% of the year, and is hence termed as *60% dependable flow* (Q_{60}). Q_{100} (*i.e.* 100% dependable flow) for a perennial river can, thus, be read out easily from such a curve. Q_{100} for an *ephmeral* or for an *intermittent river* shall evidently be zero.

A flow-duration curve represents the cumulative frequency distribution, and can be considered to represent the stream flow variation of an average year. Such a curve can be plotted on an ordinary arithmetic scale paper, or an semi-log or log-log paper. The following *characteristics of flow-duration curves* have been noticed.

(1) The slope of the flow-duration curve depends upon the interval of data used. Say for example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same river. This happens due to smoothening of small peaks in monthly data.

(2) The presence of a reservoir on a stream upstream of the gauging point will modify the flow-duration curve for the stream, depending upon the reservoir-regulation effect on the released discharges. A typical reservoir regulation effect on flow duration curve is shown in Fig. 18.7 (*c*).

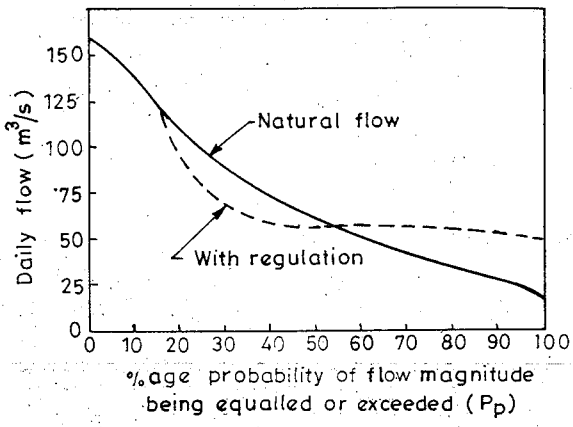


Fig. 18.7. (*c*) Reservoir regulation effect on the flow-duration curve of a stream.

(3) The flow-duration curve, when plotted on a log probability paper, is found to be a straight line at least over the central region, as shown in Fig. 18.7 (*c*). From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.

(4) The flow-duration curve plotted on a log-log paper (Fig. 18.7 (*b*)) is useful in comparing the flow characteristics of different streams. Say for example, a steep slope of the curve indicates a stream with a highly variable discharge ; while a flat slope of the curve indicates a small variability of flow and also a slow response of the catchment to the rainfall. A flat portion on the lower end of the curve indicates considerable base flow. A flat portion on the upper end of the curve is typical of river basins having large flood plains, and also of rivers having large snowfall during a wet season.

(5) The chronological sequence of occurrence of the flow gets hidden in a flow duration curve. A discharge of say 500 m^3/s in a stream will, thus, have the same percentage probability (P_p), irrespective of whether it occurred in January or June. This aspect, a *serious handicap of such curves*, must be kept in mind while interpreting a flow-duration curve.

Flow-duration curves find a considerable use in water-resources planning and development activities. Some of their important uses are indicated below :

(*i*) For evaluating dependable flows of various percentages, such as 100%, 75%, 60%, etc. in the planning of water-resources engineering projects.

(*ii*) In evaluating the characteristics of the hydropower potential of a river.

(*iii*) In comparing the adjacent catchments with a view to extend the stream flow data.

(*iv*) In computing sediment load and dissolved solids load of a stream.

(*v*) In the design of drainage systems, and

(*vi*) in flood control studies.

18.5. Fixing the Reservoir Capacity for the Computed Value of the Dependable Yield of the Reservoir Catchment

After deciding the dependable yield for the proposed reservoir (tank), the reservoir capacity is decided as follows:

The water demand (annual of course) is computed by estimating the *crop water requirement* (including transit losses) and any other water demand required to meet the water supply needs, or the downstream commitments of water release, if any. Reservoir losses @ about 15% of the water demand is then added to obtain the *live* or the *net storage* required to meet the given demand. *Dead storage* is now added to this live storage to obtain the *gross storage* required to meet the demand. The *reservoir capacity*, however, cannot exceed the *catchment yield* (inflow into the reservoir), and hence the reservoir capacity is fixed at a value which is lesser of the value of the assessed gross storage required to meet the demand; and ii) the assessed dependable yield for the reservoir site. The full tank level (FTL) or the full reservoir level (FRL) is finally computed from the elevation-capacity curve.

The *dead storage* level or dead storage required in the above computation is usually fixed at higher of the following values:

(*a*) dead storage = rate of silting x Life of the reservoir

(*b*) dead storage = 10% of gross storage or net water demand.

(*c*) dead storage level being equal to the full supply level of the off taking canal at the tank site

Example 18.5. *The lowest portion of the capacity-elevation curve of a proposed irrigation reservoir, draining 20 km^2 of catchment, is represented by the following data:*

Elevation in m	*Capacity in ha.m*
RL 600	*24.2*
602	*26.2*
604	*30.3*
606	*36.8*

The rate of silting for the catchment has been assessed to be 300 m^3/km^2/year. Assuming the life of the reservoir to be 50 years, (a) compute the dead storage, and the lowest sill level (LSL), if the main canal is 6km long with a bed slope of 1 in 1000, and the canal bed level at the tail end is at RL 594.5 m. The FSD of the canal at the head is 80 cm. The crop water requirement is assessed as 250 ham.

(b) If the dependable yield of the catchment is estimated to be 0.29m, what will be the gross capacity of the reservoir ?

Solution. The *dead storage* is first of all computed as maximum of the following three values :

(*a*) dead storage = rate of silting × life of the reservoir

$$= 300\ \text{m}^3/\text{km}^2/\text{year} \times \underset{\text{(C.A.)}}{20\ \text{km}^2} \times \underset{\text{(Life)}}{50\ \text{year}} = 300000\ \text{m}^3$$

$$= 30\ \text{ha.m.} \qquad ...(i)$$

(*b*) dead storage

= 10% × net water demand or crop water requirement = 10% × 250 ha.m

= 25 ha.m ...(*ii*)

(*c*) Dead Storage Level = FSL of canal at headworks

$$= 594.5 + (6 \times 1000)\frac{1}{1000} + 0.8 = 601.4 \text{ m}$$

Dead Storage Capacity at RL 601.4 m is interpolated

$$= 24.2 + \frac{(26.2 - 24.2) \text{ ham}}{RL\ (602 - 600)} \times RL\ (601.4 - 600) = 24.2 + \frac{2 \times 1.4}{2}$$

$$= 24.2 + 1.4 \text{ ham} = 25.6 \text{ ham} \qquad ...(iii)$$

The dead storage is fixed at maximum of the three values obtained at (*i*), (*ii*) and (*iii*) above, *i.e.* 30 ham, 25 ham, and 25.6 ham. Hence, choose the dead storage at **30 ham. Ans.**

The *LSL* (x) corresponds to 30 ham capacity, which is computed as :

$$30 = 26.2 \text{ ham} + \frac{(30.3 - 26.2) \text{ ham}}{RL\ (604 - 602)} \times RL\ (x - 602)$$

or $$3.8 = \frac{4.1}{2}(x - 602)$$

$$(x - 602) = \frac{3.8 \times 2}{4.1} = 1.85$$

$$x = RL\ \mathbf{603.85\ m.}$$

The lowest sill level *i.e.* dead storage level is thus fixed at **RL 603.85 m. Ans.**

(*b*) Net water demand = Crop water requirement including transit losses = 250 ham

Reservoir losses = 15% × 250 ham = 37.5 ham

Live storage required

to meet the given demand = (250 + 37.5) ham = **287.5 ham**

Dead storage = 30 ham (computed above)

∴ Gross reservoir storage required

= Live storage + Dead storage = 287.5 + 30 = **317.5 ham**

Dependable yield = 0.29 m (depth) = 0.29 m × (20×10^6 m^2) = **580 ham**

The gross capacity of the reservoir is fixed at the lesser of the gross storage required to meet the demand (*i.e.* 317.5 ham) and the dependable yield (*i.e.* 580 ha.m.). Hence, the reservoir capacity = **317.5 ham. Ans.**

18.6. Relation between Inflow, Outflow, and storage Data for a Reservoir

The *inflow* to the reservoir and the *outflow* from the reservoir are the only two factors which govern the storage capacity of a reservoir. Since the inflow to the reservoir is variable, water is stored in the reservoir to cater to the required outflow from the reservoir, particularly during the critical periods in non-monsoon season. Naturally, if more outflow is required, more capacity has to be provided.

As a matter of fact, after assessing the monthly or annual inflows into the reservoir and representing it by the mass inflow curve, the demand pattern is specified. The reservoir is then usually designed to meet this specified demand, represented by the mass outflow curve.

The reservoir capacity, the reservoir inflow, and the outflow from the reservoir are governed by the storage equation, given by:

Inflow – Outflow = Increase in storage

∴ Increase in Reservoir Storage = Inflow – Outflow ...(18.9)

18.7. Fixing the Reservoir Capacity from the Annual Inflow and Outflow Data

The capacity of reservoir may be determined by determining the storage needed to accommodate the given inflow minus the given outflow, as governed by the above equation. However, this study involves numerous factors, as discussed below:

Streamflow data at the reservoir site must be known. Monthly inflow rates are sufficient for large reservoirs, but daily data may be required for small reservoirs. When the inflow data at the dam site are not known, the data at a station elsewhere

on the stream or on a nearby stream may be collected and adjusted to the dam site. The available data may sometimes be extended so as to include a really drought period.

Besides determining the streamflow data at the dam site, an adjustment has to be made for the water required to be passed from the reservoir to satisfy the prior water rights and to obey the agreements between various sharing States through which the river is passing.

Moreover, the construction of a reservoir increases the exposed area of the water surface above that of the natural stream, and thus, increases the evaporation losses. There is, thus, a net loss of water occurring due to reservoir construction, Sometimes, these losses may be so huge that the entire purpose of the reservoir may be defeated. Seepage from the reservoir may also add to the loss resulting from the reservoir.

All these factors make this study very very complex. An approximate easy solution for determining the reservoir capacity may be obtained graphically with the help of *mass curves* as explained in the next article. On the other hand, a tabular solution is necessary in order to account for all important factors. For more precise results and for all complex systems, computers may be used for programming the analysis (called operation study).

Example 18.6. *Monthly inflow rates during a low-water period at the site of a proposed dam are tabulated in Col. (2) of table 18.9. The corresponding monthly pan evaporation and precipitation at a nearby station are also tabulated in Col. (3) and Col. (4) of the same table. Prior water rights make it obligatory to release the full natural flow or 15 hectare-metres per month, whichever is minimum. If the estimated monthly demands are as given in col. (5) of table 18.9 and the net increased pool area is 400 hectares, find the required storage capacity for the reservoir. Assume pan evaporation coefficient=0.7 and also assume that only 28% of the rainfall on the land area to be flooded by reservoir has reached the stream in the past.*

Table 18.11

Month	*Inflow at dam site in hectare metres*	*Pan evaporation in cm*	*Precipitation in cm*	*Demand in hectare-metres*
(1)	*(2)*	*(3)*	*(4)*	*(5)*
January	1.2	1.8	1.3	15.8
February	0.0	1.8	1.7	14.3
March	0.0	2.6	0.6	9.6
April	0.0	10.2	0.0	4.8
May	0.0	15.4	0.0	3.5
June	0.0	1.6	1.1	3.4
July	240.0	10.8	16.1	5.0
August	480.0	11.7	16.4	5.0
September	1.0	10.8	2.2	10.0
October	0.6	9.6	0.8	15.6
November	0.5	7.8	0.0	16.8
December	0.2	2.0	0.0	16.8
Σ	723.5	101.1	40.2	120.6

Solution. Table 18.11 is extended as shown in Table 18.12. The 28% precipitation is already reaching and is included in the given inflow [Col. (2)], and hence, only 72% of the precipitation is to be included, as worked out in Col. (8). The table is otherwise self- explanatory and the monthly water drawn from the reservoir is worked out in Col. (10).

Table 18.12. Solution Table for Example 18.6

Month	*Inflow at dam site in hectare-metres*	*Pan evapo-ration in cm*	*Precipi-tation in cm*	*Demand in hectare-metres*	*Require-ment due to prior rights equal to Col. (2) or 15 ha.m. whichever is minimum*	*Evaporation in hectare-metres* $\frac{400 \times Col.\ (3)}{100} \times 0.7$ $= 2.8 \times Col.(3)$	*Precipitation in hectare metres* $\frac{400 \times Col.\ (4)}{100} \times 0.72$ $= 2.88 \times Col.(4)$	*Adjusted inflow. Col. (2) + Col. (8) – Col. (6) – Col. (7) in hectare metres*	*Water required from storage in ha. m. Col. (5) – Col. (9) (only +ve values)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*
January	1.2	1.8	1.3	15.8	1.2	5.03	3.74	(–) 1.29	17.09
February	0.0	1.8	1.7	14.3	0.0	5.03	4.9	(–) 0.13	14.43
March	0.0	2.6	0.6	9.6	0.0	7.28	1.73	(–) 5.55	15.15
April	0.0	10.2	0.0	4.8	0.0	28.56	0.0	(–) 28.56	33.36
May	0.0	15.4	0.0	3.5	0.0	43.1	0.0	(–) 43.1	46.60
June	0.0	16.6	1.1	3.4	0.0	46.4	3.17	(–) 43.23	46.63
July	240.0	10.8	16.1	5.0	15.0	30.3	46.4	(+) 241.1	Nil
August	480.0	11.7	16.4	5.0	15.0	32.8	47.3	(+) 479.5	Nil
September	1.0	10.8	2.2	10.0	1.0	30.3	6.34	(–) 23.96	33.96
October	0.6	9.6	0.8	15.6	0.6	26.9	2.31	(–) 24.39	39.99
November	0.5	7.8	0.0	16.8	0.5	21.8	0.0	(–) 21.8	38.60
December	0.2	2.0	0.0	16.8	0.2	5.6	0.0	(–) 5.6	22.40
Σ	723.5	101.1	40.2	120.6	33.5	283.1	116.49		308.21

Finally, their summation (*i.e.* **308.21 hectare-metres**) works out to be the required storage capacity of the reservoir. **Ans.**

18.8. Fixation of Reservoir Capacity with the Help of Mass Curves of Inflow and Outflow

After the flow hydrographs for the stream at the dam site have been plotted for a large number of years (say 25 to 30 years), the required storage capacity for a reservoir with a given outflow pattern can be approximately calculated with the help of mass curves. A hydrograph is a plot of discharge vs. time, while a *mass curve* is a plot of accumulated flow vs. time. The area under the hydrograph between times $t = 0$ and $t = t$ will represent nothing but the accumulated flow up to the time t, and hence, the ordinate of the mass curve at time *t*.

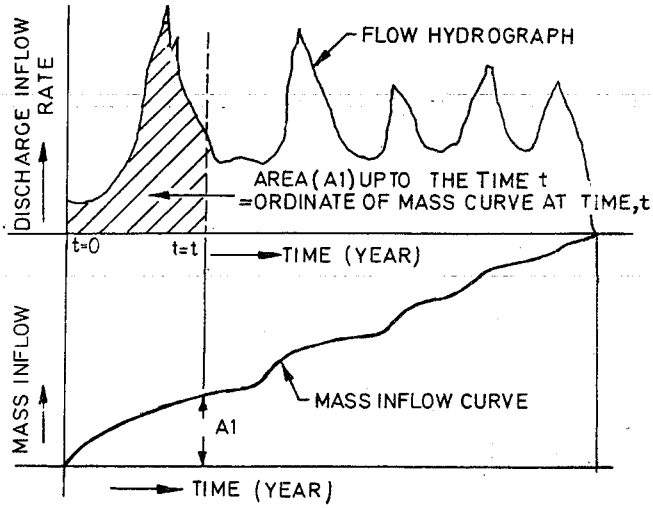


Fig. 18.8 (*a*) and (*b*).

A typical annual inflow hydrograph is shown in Fig. 18.8 (*a*), and the mass curve for this inflow hydrograph is plotted in Fig. 18.8 (*b*). The area under the first curve upto a time (*t*) is equal to the ordinate of the second curve at the same time (*t*). Adjustments for the scales and units of the two curves must be made while plotting.

It is evident that a mass curve will continuously rise, as it is the plot of the accumulated inflow. Periods of no inflow would be represented by the horizontal lines on the mass inflow curve. To differentiate such a mass curve of runoff from the mass curve of rainfall, this mass curve is usually called as the **flow mass curve** and is an integral of the flow hydrogrph.

The mass curve may also be called the *ripple diagram.* The slope of the mass curve at any time is a measure of the inflow rate at that time.

After the inflow mass curve has been plotted, the mass curve of demand may also be plotted by accumulating the required outflow. *If a constant rate of withdrawal is required from the reservoir, the mass curve of demand will be a straight line having a slope equal to the demand rate.* Demand curves or demand lines are generally straight lines (representing uniform withdrawal) although, in practice, they may be curved also [See Fig. 18.9 (*a*) and (*b*)].

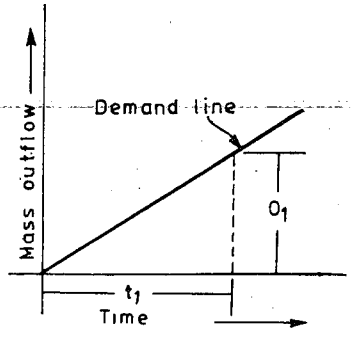


Fig. 18.9 (*a*)

O_1/t_1 = Slope of the line = Demand rate

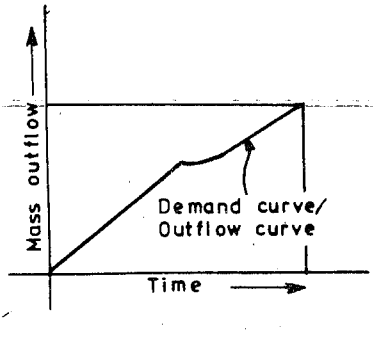


Fig. 18.9 (*b*).

Determining Reservoir Capacity for a Given Demand

The mass curve of inflow and the demand line can be used to determine the required storage capacity. In Fig. 18.10, it is evident that the demand lines drawn tangent to the high points A_1, A_2, A_3... of the mass curve, represent the rate of withdrawal from the reservoir. Assuming the reservoir to be full whenever the demand line intersects the mass curve (points F_1, F_2 ...), the maximum departure (B_1C_1, B_2C_2...) between the two curves represents the reservoir capacity just required to satisfy the demand.

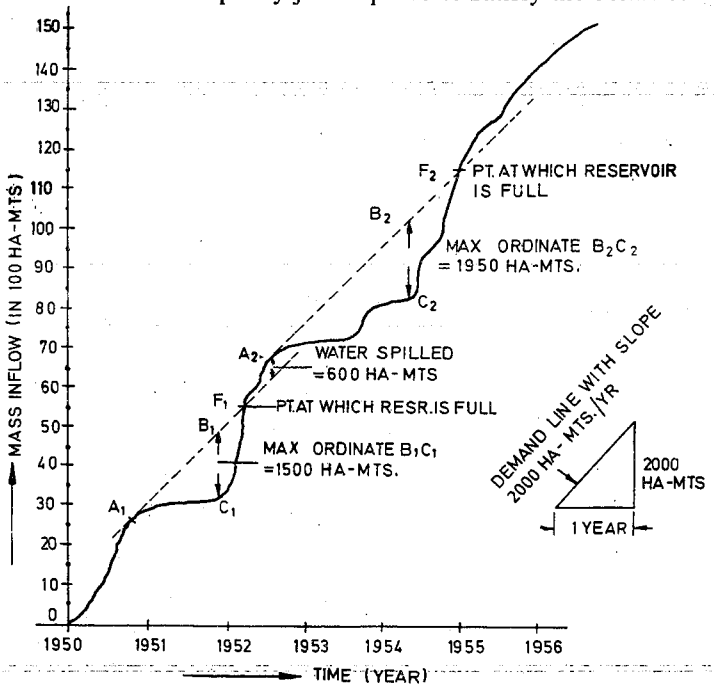


Fig. 18.10.

Fig. 18.10 has been drawn for a demand of 2,000 hectare- metres/year. The biggest departure ordinate (*i.e.* the maximum of B_1C_1, B_2C_2...) works out to be 1,950 hectare-metres/year, which represents the required storage capacity for the reservoir.

The vertical distance between the successive tangents A_1B_1, and A_2B_2, etc., represent the water wasted over the spillway. The spillway must have sufficient capacity to discharge this flood volume.

For Fig. 18.10, the spillway capacity works out to be 600 hectare-metres, and the reservoir capacity as 1,950 hectare-metres. It can also be observed that:

(*i*) Assuming the reservoir to be full at A_1, it is depleted to 1,950 – 1,500 = 450 ha-m at C_1 and is again full at F_1.

(*ii*) The reservoir is full between F_1 and A_2, and the quantity of water spilled over the spillway is equal to 600 ha-m.

(*iii*) From A_2, the water starts reducing in the reservoir till it becomes fully empty at C_2.

(*iv*) The water again starts collecting in the reservoir and it is again full at F_2.

Note 1. It may also be noted that a demand line, when extended, must intersect the mass curve. If it does not, the reservoir will not refill.

Note 2. When the demand curve is not a straight line, then the two mass curves are superimposed over each other in such a way that their origins and axis coincide. The larger ordinate between the two, gives the required storage capacity, as shown in Fig. 18.11.

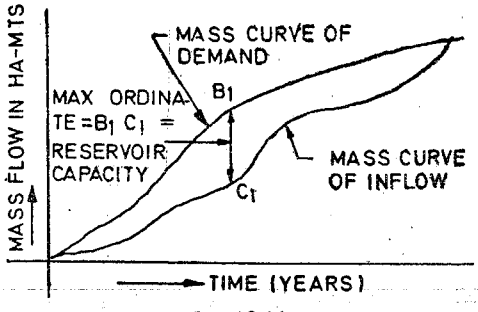


Fig. 18.11.

Fixing the Demand for a Reservoir of a Given Capacity

In the previous article, we have explained as to how the reservoir capacity can be determined for a given demand. The reverse, *i.e.* fixing the demand for a given reservoir capacity, may also be done with the help of mass curve of inflow.

In this case, the tangents are drawn to the high points (A_1, A_2, ...) of the mass inflow curve in such a way that the maximum departure from the mass curve is equal to the reservoir capacity. The slopes of the lines so drawn represent the demand rate which can be obtained with this capacity during different periods. The minimum value of these slopes will represent the withdrawal rate, which can certainly be obtained from the given reservoir, and will, thus, represent its *firm yield.*

For example in Fig. 18.12, *a mass inflow curve is given. It is further required to find out the possible yields and the safe yield for a reservoir capacity of 750 ha-m.* To determine these values, we shall proceed as follows :

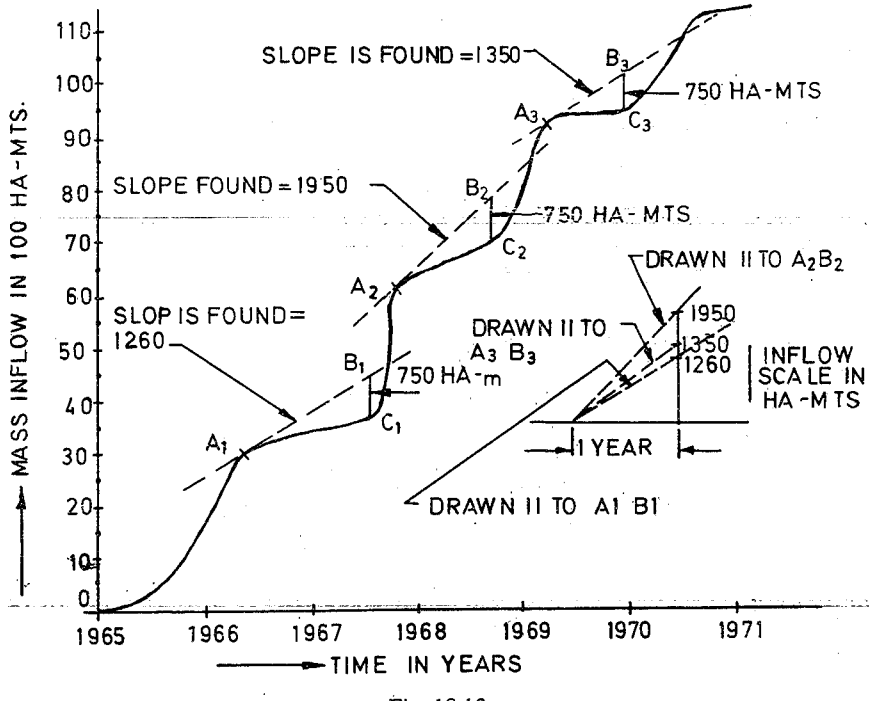


Fig. 18.12.

First of all, the high points A_1, A_2 and A_3 are marked on the mass inflow curve. The points B_1, B_2, and B_3 are determined in such a way that their maximum departure from the curve is equal to 750 ha-m. The tangent lines A_1B_1, A_2B_2 and A_3B_3 are then drawn. The slopes of these lines are determined and are found to be 1,260, 1,950, 1,350

ha-m/year, respectively. These values represent the possible yields in different periods. *The minimum of them, i.e.* 1,260 ha-m/year represents the safe yield or the firm yield of the reservoir. **Ans.**

Example 18.7. *Annual runoff in terms of depth over the catchment area of 1675 sq.km. of a reservoir is given below :*

Year	*1962*	*63*	*64*	*65*	*66*	*67*	*68*	*69*
Runoff (cm)	*98*	*143.5*	*168.3*	*94*	*95.3*	*152.4*	*110*	*131.3*

Draw the flow mass diagram. What is the average yield from the catchment? What should be the live storage capacity of the reservoir to use the source fully? If the dead storage is 20% of the live storage, what is the gross storage? Mark the filling and emptying periods on the mass curve. (*Bhopal University, 1980*)

Solution. The cumulative runoff values are worked out in Col. 4 of table 18.13, and they are plotted against the values of corresponding years (col. 1) of the same table, so as to obtain the desired mass diagram [Fig. 18.13].

Table 18.13

Year	*Yearly runoff (cm)*	*Runoff as volume in* $M.m^3 = \frac{Col.\ (2)}{100} \times 1675$	*Cumulative runoff as volume in* $M.m^3$
(1)	*(2)*	*(3)*	*(4)*
1962	98	1642	1642
63	143.5	2404	4046
64	168.3	2819	6865
65	94	1575	8440
66	95.3	1596	10,036
67	152.4	2553	12,589
68	110	1842	14,431
69	131.3	2199	16,630
	$\Sigma = 992.8$	16630	

The **average annual yield** of the catchment is the arithmetic mean of the given annual yields, and is equal to

$$= \frac{992.8}{8} = 124.1 \text{ cm of runoff}$$

$$= 1.241 \text{ m} \times CA \text{ in m}^2 \qquad \text{(volume of runoff)}$$

$$= 1.241 \text{ m} \times (1675 \times 10^6) \text{ m}^2$$

$$= 2078.68 \text{ Mm}^3; \text{ say } \mathbf{2079\ Mm^3} \quad \textbf{Ans.}$$

Now, to utilise the source fully, there should not be any spilling over of the water, and the yearly demand should be equal to the average yield *i.e.* 2079 Mm^3.

To determine the required reservoir capacity to meet this demand rate, a line is drawn from the high point A_1, parallel to this demand rate, as shown in Fig. 18.13; and the maximum departure of this line from the mass curve is read out as $B_1C_1 = 1065\ Mm^3$.

Hence, the required reservoir capacity is 1065 Mm^3. **Ans.**

Physically speaking, the reservoir will be empty at trough pts C_1 and C_2, and full at ridge point A_1.

Alternatively, the mass demand line can be plotted, and both curves extended back up to the origin O, as shown in Fig. 18.14.

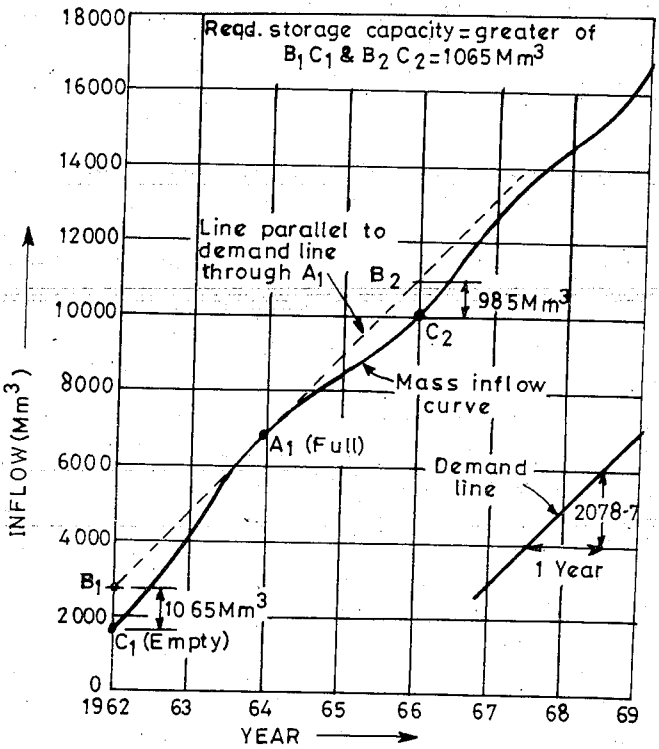


Fig. 18.13.

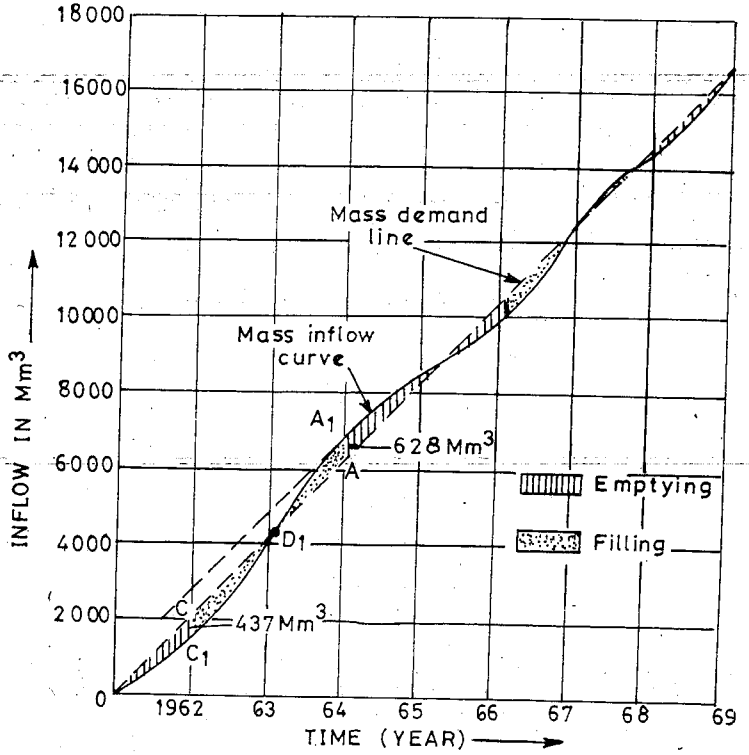


Fig. 18.14.

It can be seen from this curve, that from O to C_1, the slope of the inflow curve is less than that of the demand curve, indicating that the inflow is less than the outflow, and the reservoir is *emptying*. From C_1 to D_1, the slope of the inflow curve is in excess of the demand, and reservoir is *filling*. The vertical intercept $C_1 C$, from the inflow curve at C_1 to the demand curve at C, represents the initial storage to meet the demand from O to C_1. At D_1, the reservoir level is the same as at O. After D_1, and upto A_1, the slope of the inflow curve is steeper than that of the demand curve, and as such, the reservoir is still rising. Point A_1 represents the full reservoir level. Vertical intercept A_1A represents the storage between the initial water level corresponding to the point O and the full reservoir level. The total minimum storage capacity of the reservoir to meet the demand is thus given by the vertical intercepts C_1C and A_1A.

Max. withdrawl from storage + Max. stored in storage

$= 437 + 628 = 1065$ mcm. **Ans.**

The emptying and filling processes are also shown in Fig. 18.14 ; filling will occur when the slope of the inflow curve is more than that of the demand curve, and *vice versa.*

18.9. Fixation of Reservoir Capacity Analytically using Sequent Peak Algorithm

The *sequent peak algorithm* is a simple and straight forward analytical procedure, for computing reservoir capacity, and is used as an excellent alternative to the mass curve method of determining reservoir capacity.

In the mass curve analysis, the reservoir is assumed to be full at the beginning of the dry period, and storage required to pass the dry period is estimated. If the mass curve contains only one ridge point, and if there is no well defined subsequent trough point, it may become necessary to repeat the given data for one more cycle to arrive at the desired storage determination. Also, the demand rate is usually not a straight line (as assumed in mass curve analysis), since the demand (out flow) generally becomes non-uniform due to seasonal variations in the demand. *The sequent peak algorithm technique helps us to device simple mathematical solution to the problem of computing the reservoir capacity.*

Sequent peak algorithm (Fig. 18.15) is a plot between time (say, in months) on X-axis, and cumulative inflow minus cumulatives outflow on Y-axis. The quantity taken on Y-axis, corresponding to each month equals to [Σ Inflow $-$ Σ outflow] = [Σ (Inflow-Outflow]. This value is also called *cumulative net inflow.*

The $+$ *ve* values of cumulative net inflow, representing cumulative surplus of inflow will be plotted above X-axis, while its negative values, representing cumulative deficit of inflow, will be plotted below x-axis.

The obtained plot will consist of peaks and troughs, as shown in Fig. 18.15.

The first ridge point A_1 in this plotting (*i.e.* first peak of cumulative net surplus) is called the *first peak* ; while subsequent ridge points A_2, A_3, A_4, etc., are called *sequent peaks.*

Similarly, the first trough point B_1 is called the first trough, while subsequent trough points B_2, B_3, etc. may be called the *sequent troughs.*

The difference between the first peak and first trough (height $A_1 B_1$) will, in this plot, evidently, represent the reservoir storage required under normal inflows, and

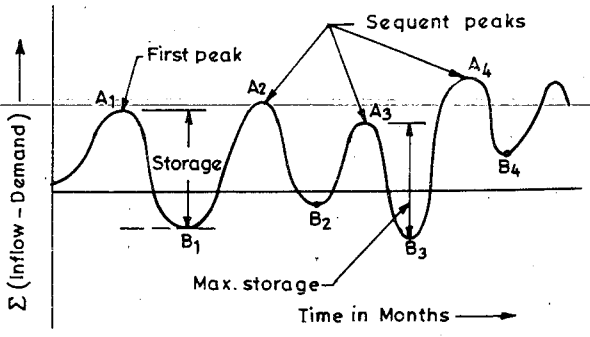


Fig. 18.15. Sequent Peak Algorithm.

is called the *normal storage* ; whereas, the maximum difference between any sequent peak and the just following trough will represent the *maximum storage* required for the reservoir.

The normal and maximum storage through sequent peak algorithm is calculated as follows :

(1) Convert the monthly inflows into the volume units for the period of the available data.
(2) Estimate the monthly volumes of all the outflows from the reservoir. This should include losses from evaporation, seepage, and other losses.
(3) Compute the cumulative values of Inflows.
(4) Compute the cumulative values of outflows.
(5) Compute the values of cumulative inflow minus cumulative outflow ; *i.e.* [Σ Inflow – Σ Outflow].
(6) Plot a graph by taking months (time) on X-axis, and $\Sigma (I - 0)$ of step (5) on Y-axis, on an ordinary graph paper.
(7) The data will plot peaks and troughs. The second and subsequent peaks are called sequent peaks.
(8) The maximum difference between any sequent peak and the just following trough is the maximum storage required for the reservoir. The difference between the first peak and the trough following it, is the storage required under normal inflows.

The method will become more clear on solving example 18.8.

Example 18.8. *Monthly inflows at a proposed reservoir site for a drought period of 15 months are given along with targetted demands (found from a working table) in the table below. Compute the storage required by plotting sequent peak algorithm.*

Table 18.14

Month	*June*	*July*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*	*Jan*	*Feb*	*March*	*April*	*May*	*June*	*July*	*Aug*
River Inflows ($M\,m^3$)	250	350	400	200	150	150	100	50	150	300	400	450	150	200	450
Targetted demand $M\,m^3$	150	150	200	250	350	400	250	200	150	150	100	250	350	300	100

Solution. Calculations are carried out in Table 18.15 to compute $\Sigma (I - O)$ in col (6).

Table 18.15

Month	*Inflow (I) $M\,m^3$*	*Outflow (O) $M\,m^3$*	*Cumulative Inflow $\Sigma I\,(M\,m^3)$*	*Cumulative Outflow $\Sigma O\,(M\,m^3)$*	$(\Sigma I - \Sigma O)$
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
June	250	150	250	150	+ 100
July	350	150	600	300	+ 300
Aug	400	200	1000	500	+ 500
Sept	200	250	1200	750	+ 450
Oct	150	350	1350	1100	+ 250
Nov	150	400	1500	1500	0
Dec	100	250	1600	1750	(–) 150
Jan	50	200	1650	1950	(–) 300
Feb	150	150	1800	2100	(–) 300
March	300	150	2100	2250	(–) 150
April	400	100	2500	2350	+ 150
May	450	250	2950	2600	+ 350
June	150	350	3100	2950	+ 150
july	200	300	3300	3250	+ 50
Aug	450	100	3750	3350	+ 400

To calculate values of $\Sigma (I - O)$, values of ΣI (cumulative inflow) are written in col (4), and values of ΣO (cumulative outflow) are written in col (5).

Values of $(\Sigma I - \Sigma O) = \Sigma (I - O)$ are new calculated by subtracting values of col (5) from values of col (4). These values are written in col (6) with their $+ve$ or $-ve$ sign, representing excess & cumulative deficit, respectively. When values of $(\Sigma I - \Sigma O)$ from col (6) are plotted on y-axis along with corresponding values of month (col 1) taken on x-axis, a plot containing peaks and troughs will be obtained, as shown in Fig. 18.16, called sequent peak algorithm. The difference between first peak and the trough following it, is found to be **800 M m^3**, which represents the reqd. storage capacity. **Ans.**

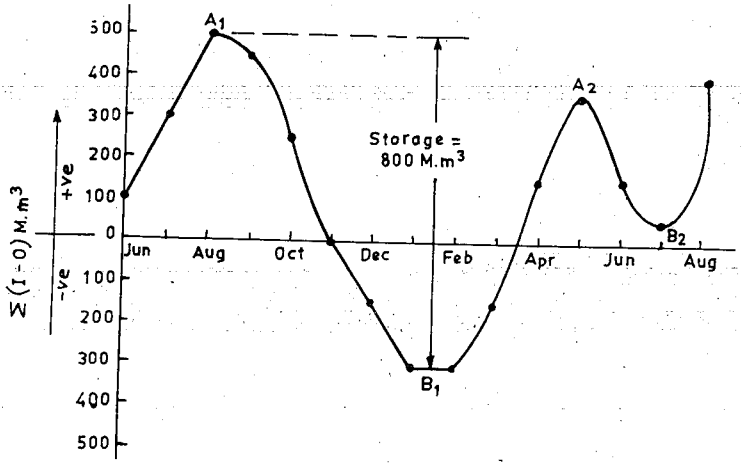


Fig. 18.16. Sequent peak algorithm for Example 18.8.

Alternatively, the values of $\Sigma (I - O)$ can be more easily computed by first computing $+ve$ and $-ve$ values of $(I - O)$, as in col (4) & (5) of Table 18.16 ; and then computing their summation values, *i.e.* values of $\Sigma (I - O)$ as in col (6) and (7) of Table 18.6. $+ve$ values represent cumulative excess inflow ; while $-ve$ values represent cumulative deficit. The maximum value out of all the values of Col (6) and (7) will represent the minimum storage required to accommodate the surplus (or to supply the deficit). This method thus, avoids the necessity of plotting peaks & troughs (curve) and the value of reqd storage capacity can be computed easily and mathematically, which procedure can even be programmed in a digital computer to help in estimation of required storage capacity. In present calculations, this peak value in col (6) & (7) is found to be **800 M.m^3. Ans.**

Table 18.16

Month	*Inflow I*	*Outflow (O)*	$(I - O)$		$\Sigma (I - O)$	
			+ ve values	*– ve values*	*Cumulative*	*Cumulative*
	M m^3	*M m^3*	*Excess (M m^3)*	*Deficit (M m^3)*	*excess (M m^3)*	*deficit (M m^3)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
June	250	150	100		100	
July	350	150	200		300	
Aug	400	200	200		500	
Sept	200	250		50		50
Oct	150	350		200		250
Nov	150	400		250		500
Dec	100	250		150		650
Jan	50	200		150		**800***
Feb	150	150	0		0	(Peak value
March	300	150	150		150	in col 6 & 7)
April	400	100	300		450	
May	450	250	200		650	
June	150	350		200		200
July	200	300		100		300
Aug	450	100	350		350	

* Max. of all the values in col (6) & (7)

Example 18.9. *Solve Example 18.7 analytically without using mass curve.*

Solution. The data given in example 18.7 is used in Table 18.17 to compute $(I - O)$ values in col (4) and (5) [– *ve i.e.* deficit values in col (4), and + *ve i.e.* Excess values in col (5)]. The values of $\Sigma (I - O)$ are finally computed in col (6) & (7). as shown :

Table 18.17

Year	*Inflow (I)* Mm^3	*Outflow (O)* Mm^3	$I - O$		$\Sigma (I - O)$	
			– ve	*+ ve*	*Cumulative*	*Cumulative*
			Deficit Mm^3	*Surplus* Mm^3	*Deficit* Mm^3	*Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1962	1642	2079	437		437	
63	2404	2079		325		325
64	2819	2079		740		1065*
65	1575	2079	504		504	
66	1596	2079	483		987	
67	2553	2079		474		474
68	1842	2079	237		237	
69	2199	2079		120		120

Out of all the values of col. (6) and (7), the max. value is 1065 Mm^3, which represents the min. storage required to accommodate this 1065 Mm^3 surplus water entering during the years 1963 and 1964. **Ans.**

Gross storage reqd. = Dead storage + Live storage

= 20% of Live storage + Live storage

= 1.2 Live storage = 1.2×1065 Mm^3 = 1278 Mm^3 **Ans.**

Example 18.10. *The yield of water in Mm^3 from a catchment area during each successive month is given in the table below :*

1.4	*2.1*	*2.8*	*8.4*	*11.9*	*11.9*
7.7	*2.8*	*2.52*	*2.24*	*1.96*	*1.68*

Determine the minimum capacity of a reservoir required to allow the above volume of water to be drawn off at a uniform rate assuming that there is no loss of water over the spillway.

Solution. The total inflow of water in 12 months

= Summation of inflow values = 57.4 Mm^3.

∴ Average monthly rate at which water is withdrawn to use the inflow fully = Av. demand rate = $57.4/12 = 4.78$ Mm^3.

Now, to draw the mass curve of inflow, the cumulative inflow values are worked out in table 18.18.

Table 18.18

Month	*Yield* (Mm^3)	*Cumulative yield* (Mm^3)
1	1.4	1.4
2	2.1	3.5
3	2.8	6.3
4	8.4	14.7
5	11.9	26.6
6	11.9	38.5
7	7.7	46.2
8	2.8	49.0
9	2.52	51.52
10	2.24	53.76
11	1.96	55.72
12	1.68	57.40

The mass inflow curve is now plotted, as shown in Fig. 18.17. A line parallel to the demand rate line is now drawn through the high point A_1 on the inflow mass curve, as shown. The maximum departure between the inflow mass curve and this line, *i.e.* B_1C_1, gives the min. storage capacity reqd. This value is read out as 20.78 Mm^3. **Ans.**

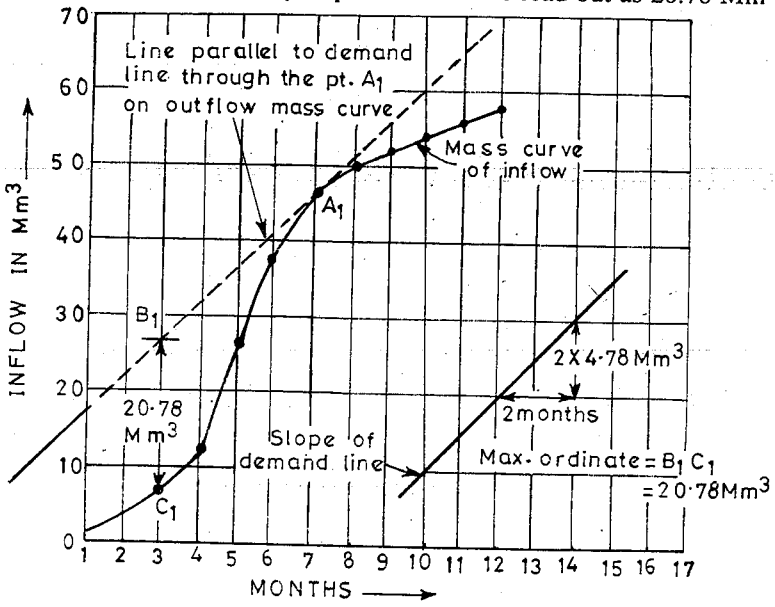


Fig. 18.17.

Analytically, the solution to the problem can be worked out, as shown in table 18.19.

Table 18.19

Month	*Inflow* Mm^3	*Outflow (Demand)* Mm^3	*Deficit* Mm^3	*Surplus* Mm^3	*Cumulative Deficit* Mm^3	*Cumulative Surplus* Mm^3
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1	1.4	4.78	3.38			
2	2.1	4.78	2.68			
3	2.8	4.78	1.98		8.04	
4	8.4	4.78		3.62		
5	11.9	4.78		7.12		
6	11.9	4.78		7.12		
7	7.7	4.78		2.92		20.78
8	2.8	4.78	1.98			
9	2.52	4.78	2.26			
10	2.24	4.78	2.54			
11	1.96	4.78	2.82			
12	1.68	4.78	3.10		12.70	

The highest value in col. (6) and 7 (as no spilling is allowed), is 20.78 Mm^3, which gives the min. required reservoir storage. **Ans.**

Example 18.11. *The runoff data for a river during a lean year are given below :*

Month	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sept*	*Oct*	*Nov*	*Dec*
River flow in $10^6 m^3$	140	27	35	26	16	48	212	180	116	92	67	37

What is the maximum uniform demand that can be met? What is the storage capacity required to meet this demand? What minimum initial storage is necessary? When does the reservoir become empty?

Solution. Total inflow in the year = Summation of the given monthly discharges

$= 996$ M. m^3.

Average monthly rate at which water can be withdrawn to avoid any wastage *i.e. max average monthly rate*

$$= \frac{996}{12} = 83 \text{ Mm}^3 \quad \textbf{Ans.}$$

Now, to determine storage capacity etc. we carry out the computations in table 18.20.

Table 18.20

Month end	*Monthly Inflow in reservoir in Mm^3.*	*Monthly outflow from reservoir in Mm^3.*	*Monthly deficit in reservoir (to be supplied from storage) in Mm^3*	*Monthly surplus in reservoir in Mm^3*	*Consecutive cumulative deficit in Mm^3*	*Consecutive cumulative surplus in Mm^3*	*Net water available in the reservoir in Mm^3*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
Jan	140	83		57		57	(+) 57
Feb	27	83	56				
March	35	83	48				
April	26	83	57				
May	16	83	67				
June	48	83	35		263		(–) 206
July	212	83		129			
Aug	180	83		97			
Sept	116	83		33			
Oct	92	83		9		268	(+) 62
Nov	67	83	16				
Dec	37	83	46		62		0

The highest value in col. (6) and (7) is **268 Mm^3**, which represents the minimum storage capacity required to meet the demand without any spilling. **Ans.**

To compute the min. initial storage, we compute in col. (8), the net storage left in the reservoir with the above inflows and outflows. The maximum negative storage here works out to be 206 Mm^3. In order that the reservoir fully meets the demand with the above inflows & outflows, there should be no negative storage in it, and in the limiting case the max. negative storage should be equal to zero. Hence, the min. initial storage in the reservoir should be **206 Mm^3**, which will just meet the shortage created in the reservoir by *June end,* when the reservoir will become empty. **Ans.**

Example 18.12. *The storage capacity of a reservoir for a flood control project is to be determined. The estimated cost of damage if the emergency spillway is topped is Rs. 10 lacs for each event. The interest rate is 6% and the reservoir life is 50 years. Six reservoir designs of different storage capacities ; the probabilities of exceeding those capacities in any given year, and the estimated initial cost for construction of the reservoir are as follows :*

Reservior design number	*1*	*2*	*3*	*4*	*5*	*6*
Flood storage capacity in lac cubic metres	*30*	*35*	*40*	*45*	*50*	*55*
Probability of exceeding storage capacity in any given year	*0.15*	*0.10*	*0.06*	*0.04*	*0.02*	*0.01*
Estimated initial cost, in lac rupees	*25*	*30*	*35*	*40*	*45*	*50*

Determine the optimal storage capacity of the reservoir.

(A.M.I.E. 1989)

Solution. The required computations are carried out in table 18.21 below.

Table 18.21

Reservior Design No.	*Reservior Capacity in lakh cum*	*Probability of exceeding storage capacity*	*No. of times the flood is likely to exceed the capacity in 50 yrs.*	*Damages caused due to flood exceeding capacity in Rs. Lakh = Rs. 10 lakh × Col (4)*	*Extra benefit over proposal No. 1 of lowest storage as datum; Rs. 75 lakh – col (5)*	*Imtial cost of each proposal in Rs. Lakh*	*Extra costs involved over proposal 1 as datum, in Rs. Lakh Col. 7 – Rs. 25 lakh*	*Net additional benefit in Rs. lakh; col. (6) – col. (8)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
1	30	15%	7.5	75	—	25	—	—
2	35	10%	5	50	25	30	5	20
3	40	6%	3	30	45	35	10	35
4	45	4%	2	20	55	40	15	40
5	50	2%	1	10	65	45	20	45
6	55	1%	0.5	5	70	50	25	45

The above table computes the damages caused by overflooding for each proposal, the max. damage of Rs. 75 lakhs being for proposal 1 - having min. reservoir capacity; and the min. damage of Rs. 5 lakh being for proposal 6 – having max. storage capacity.

The additional benefits thus provided by proposals 2 to 6 over proposal 1 are computed in col (6) as 75 lakhs – col (5). Extra costs are also worked out in col (8).

These calculations reflect that in proposal 2, we spend extra Rs. 5 lakh (over proposal 1) and get a benefit of 25 lakhs (over proposal 1 of course) ; *i.e.* a net benefit of Rs. 20 lakh. Similarly, in proposal 3, we spend extra Rs. 10 lakhs & get a benefit of Rs. 45 lakhs over proposal 1; *i.e.* a net benefit of Rs. 35 lakhs. Similarly, we find that our net benefit increases up to proposal 5, and afterwards, as in proposal 6, it becomes equal to that in proposal 5. Hence, the increase in reservoir capacity up to proposal 5 gives us increasing net returns, but thereafter we don't get any extra net returns. Investing more, beyond proposal 5, would, therefore, not be optimal, and thus, proposal 5 can be considered as the most optimum design.

Hence, the optimum storage capacity of the reservoir will correspond to that of proposal 5, *i.e.* **50 lakh m^3. Ans.**

18.10. Estimation of Demands and Optimal Reservoir Operations

In the previous articles, it was explained as to how the reservoir capacity can be determined for satisfying a certain given downstream demand or *vice versa.* This demand may be constant or may vary throughout the year.

The demand pattern mainly depends upon the purpose for which the reservoir has been constructed. Hence, it has to be different for different types of reservoirs. Moreover, the demands are man made, and should be adjusted in such a way that the maximum benefits can be obtained with the minimum cost. If by reducing the demands slightly, a lot of expenditure can be saved, then we must go in for that. This is known as *optimal planning* for dam reservoirs, and involves economic considerations, discussed afterwards.

Demand patterns for various types of reservoirs are explained below :

(*i*) *Single Purpose Conservation Reservoir.* A single purpose conservation reservoir shall store water only to meet one particular demand, such as water supply or irrigation or hydroelectric power generation, etc. In all such cases, the demand for water can be predicted with a fair degree of accuracy for different parts of the year, and the storage capacity can be worked out, as explained earlier. The storage capacity so worked out should be otherwise feasible and should be justified on cost benefit considerations.

(*ii*) *Single Purpose Flood Control Reservoir.* In case of a single purpose flood control reservoir, there is no specific downstream demand at all, except to satisfy that the downstream release should not exceed the safe carrying capacity of the channel. Since the effect of constructing a flood control reservoir is to moderate and reduce the flood peaks by absorbing certain volume of flood, and then gradually releasing it when the flood subsides (Fig. 18.18), the capacity of such a reservoir should be sufficient to absorb this excess volume of flood.

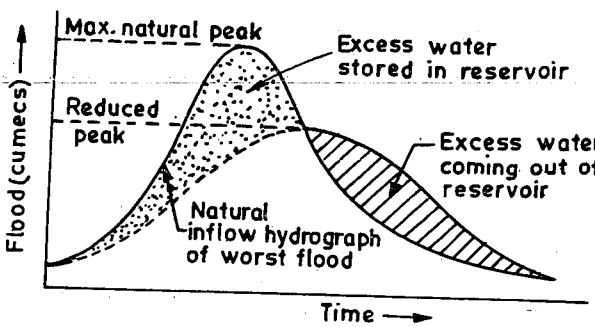


Fig. 18.18.

It, therefore, becomes evident that the capacity of such a

reservoir does not depend upon the pattern of demand but mainly depends upon the hydrograph of the worst flood that is likely to enter this reservoir and also upon the downstream permissible H.F.L. and the safe carrying capacity of the channel.

The hydrograph of the worst inflow flood can be found from the hydrological investigations, and the safe peak rate of outflow can be determined from the downstream channel conditions. The capacity of the reservoir required to moderate the inflow peak to a value equal to or less than the downstream safe peak can then be found by hit and trial method with the help of *flood routing*. Flood routing is the process by which the hydrograph of the moderated flood can be determined, and is explained a little later.

(*iii*) *Multipurpose Reservoirs.* Single purpose reservoirs are seldom constructed these days. Reservoirs are therefore, generally designed to serve more than one purpose. For example, a reservoir constructed for conserving water for irrigation may be combined with its flood control purpose. The head available may simultaneously be utilised for generating hydro-electric power. This combination of irrigation, flood control, and power is generally adopted in designing multipurpose reservoirs in India.

For proper optimal planning of such a multipurpose reservoir, a schedule of operations must be finalised from a number of tentative schedules drawn on the basis of available data and past similar experiences. The schedule which gives maximum total benefits for the various design purposes, without encroaching upon the lower or upper limits of storage, is accepted for estimation and for initial operations. This schedule may be tried and further modified on the basis of the past experience and future needs.

A multipurpose reservoir constructed on a perennial snow fed river in North India (such as Bhakra reservoir) can be operated on the following lines :

Fig. 18.19 shows the typical schedule of operations for such a reservoir. The reservoir water will normally fluctuate between minimum pool level and normal pool level (*i.e.* maximum conservation level) for satisfying irrigation needs. The minimum pool level will ensure the generation of firm power, as the water level shall not normally

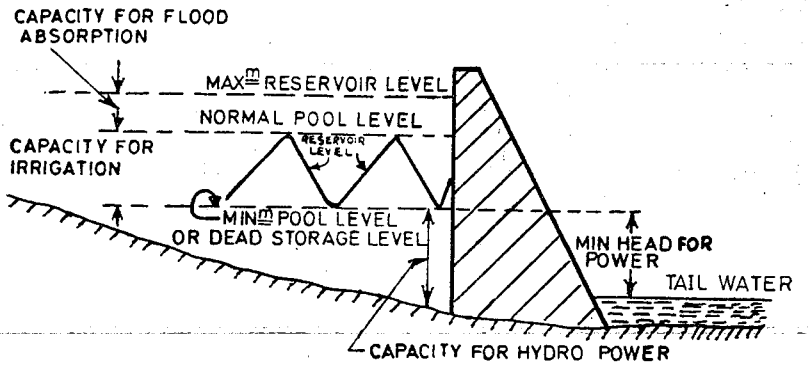


Fig. 18.19.

be allowed to go below the minimum pool level. Any serious flood which may occur after the reservoir is full, shall be absorbed between the normal pool level and the miximum pool level, at all times. As soon as the water level reaches the normal pool level, the spillway will start functioning. But the rate of water discharged over the spillway shall be in accordance with the downstream safe carrying capacity of the

channel, and hence, may be less than the rate of inflow. The water shall, then, be stored temporarily between the normal pool level and the maximum reservoir level, and discharged at a safe rate. As the flood subsides, and the excess water goes out slowly, the water level will again fall up to the normal pool level, and the reservoir will be ready to absorb another flood.

For irrigation, we generally require water from April to June for Kharif crops, and from middle of November to middle of February for Rabi crops. There are heavy rains during monsoon period of say June to middle of October. At the start of the monsoon season, the reservoir is quite depleted because of heavy outflow for Kharif crops and constant release for power generation. Thus, just before the monsoon, a large reservoir capacity is available, which not only conserves the water but also serves to control floods that may occur at this time. During the monsoon, the irrigation demand will almost be nil and water shall be released only for power generation. The reservoir level will, therefore, go on increasing steadily and will be allowed to go up to the normal pool level by the end of normal monsoon season, say middle of October. Any untimely flood occurring after the reservoir is full up to normal pool level, shall be absorbed between normal pool level and the maximum reservoir level, as explained earlier.

After the monsoon is over, the inflow is considerably reduced (almost nil) and water is constantly drawn for Rabi crops and power generation. Due to this, the reservoir level goes on falling up to about middle of February, when it is usually the lowest, somewhere near the minimum pool level (*i.e.* dead storage level). At this time, snow starts melting in the head reaches of the river, which augments the river discharge. Some winter cyclonic rains during December and January may also be helpful in augmenting the reservoir storage. From February-March onwards, there are large supplies due to melting of snow, and an equal burden is to be absorbed by the reservoir for satisfying the Kharif irrigation withdrawals. Thus, during April, May and June, the reservoir level may rise or fall slightly at slow rate; and at the start of monsoon, *i.e.* at the end of June or beginning of July, the reservoir will be quite depleted and prepared for full replenishment of its supplies.

FLOOD ROUTING OR FLOOD ABSORPTION

The hydrograph of a flood entering a reservoir, will change in shape as it emerges out of the reservoir, because certain volume of its water is stored in the reservoir temporarily and is let off as the flood subsides. The base of the hydrograph, therefore, gets broadened, its peak gets reduced, and, of course, the time of peak is delayed. The extent by which the inflow hydrograph gets modified due to the reservoir storage can be computed by a process known as *flood routing,* and more particularly as *reservoir routing* (to differentiate it from routing though 'river channels').

Since the flood protection reservoirs are generally located many km upstream of the cities which are to be saved against floods, it is sometimes necessary to route the outflow hydrograph of the reservoirs up to these downstream localities. The reservoir outflow hydrograph may then be routed through this much length of river channel, so as to obtain the final shape of the hydrograph at the affected cities. This routing, in which the stream itself acts like an elongated reservoir, is known as *channel routing* or *river routing.*

A variety of routing methods are available, and they can be broadly classified into two categories, *viz*

(*i*) *hydrologic routing ; and*

(*ii*) *hydraulic routing*

Hydrologic routing methods employ essentially the *equation of continuity ; whereas the hydraulic routing methods employ the continuity equation together with the equation of motion of unsteady flow.* The basic differential equations used in hydraulic routing are popularly known as *St. Venant equations*, and afford a better description of the unsteady flow than the hydrologic methods. We shall, however, cofine ourselves in the book to the hydrologic methods only.

18.11. Hydrologic Reservoir Routing Methods

The passage of a flood wave through a reservoir or a river reach is an *unsteady flow* phenomenon. In hydraulics, we classify it as a *gradually varied flow.* The *equation of continuity* used in all the hydrologic routing methods, as the primary equation, states that the difference between the inflow and the outflow rate is equal to the rate of change of storage ; *i.e.*

$$I - O = \frac{dS}{dt} \qquad ...(18.9)$$

where I = Inflow rate
O = Outflow rate
S = Storage

Alternatively, in a small time interval Δt, the difference between the total inflow volume and the total outflow volume is equal to the change in storage volume ; *viz.*

$$\underset{\substack{\text{(Inflow volume)}\\ I(v)}}{\bar{I}\,\Delta t} - \underset{\substack{\text{(Outflow vol.)}\\ O(v)}}{\bar{O}\,\Delta t} = \Delta S \qquad ...(18.10)$$

where $\bar{I}$ = Average inflow (rate) in time Δt
$\bar{O}$ = Average outflow (rate) in time Δt
ΔS = change in storage during the time Δt

Since $\bar{I} = \frac{I_1 + I_2}{2}$;

$\bar{O} = \frac{O_1 + O_2}{2}$;

$\Delta S = S_2 - S_1$;

where suffixes 1 and 2 denote the beginning and the end of the time interval Δt.

Eq. (18.10) can then be written as :

$$\left(\frac{I_1 + I_2}{2}\right)\Delta t - \left(\frac{O_1 + O_2}{2}\right)\Delta t = S_2 - S_1 \qquad ...(18.11)$$

The time interval Δt should be sufficiently short, so that the inflow and outflow hydrographs can be assumed to be in straight line, in that time interval. Moreover, Δt must be shorter than the time of transit of the flood wave through the reservoir or the given river reach.

The above relationship seems to be very simple, but its evaluation is not easily possible without drastic simplifying assumptions. This is because of the fact that the relations between time and rate of inflow, elevation and storage of reservoir, and

elevation and rate of outflow, cannot be expressed by simple algebraic equations. They are, respectively represented by the *inflow flood hydrograph, the elevation-storage curve, and the outflow-elevation curve.* The first two curves obviously cannot be represented by any simple equations, and the third may be represented by the spillway-discharge equation ($Q = 1.71\ LH^{3/2}$) only if, the discharge through the outlets is neglected. If the discharge though the outlets is also not neglected, then all the three curves will be unamenable to simple mathematical treatment, without drastic assumptions.

Several procedures have, however, been suggested by different investigators to solve the above basic equation (Eq. 18.11) by rearranging the components in different manners. Depending upon the different procedures adopted for solving the above basic equation, the following hydrologic methods may be used for reservoir routing :

(1) *Trial and Error method*

(2) *Modified Pul's method or Storage indication method ; and*

(3) *Goodrich method.*

The first method is discussed here, while the detailed description of other methods is available in authors another book titled "*Hydrology and Water Resources Engineering*" and may be referred to in specific needs.

18.11.1. Trial and Error Method of Reservoir Routing. Trial and Error method is widely adopted with the assistance of computers to reduce the time taken in long calculations involved in this method. This method arranges the basic routing equation (Eq. 18.11), as follows :

$$\frac{I_1 + I_2}{2} \cdot \Delta t = \frac{O_1 + O_2}{2} \cdot \Delta t + (S_2 - S_1) \qquad ...(18.11\ a)$$

The procedure involves assuming of a particular level in the reservoir at the end of the interval Δt, and computing the values on the right side of the above Eq. (18.11 *a*). The summation of $\frac{O_1 + O_2}{2} \cdot \Delta t$ and $(S_2 - S_1)$ is then compared with the known value of $\frac{I_1 + I_2}{2} \cdot \Delta t$. If the two values tally, then the assumed reservoir elevation at the end of the interval is supposed to be O.K. ; otherwise this is changed, and the process is repeated till the required matching is obtained.

This method gives quite reliable results, provided the chosen time interval (Δt) is sufficiently small, so that the mean of the outflow rates at the start and the end of the given interval may be taken as the average throughout the interval.

Procedure : The following detailed procedure may be adopted in this method, to complete the involved computations :

Data to be given : (*i*) the inflow hydrograph

(*ii*) Elevation capacity curve or Elevation area curve

(*iii*) Elevation outflow curve.

Steps involved in computations

(*i*) Divide the inflow flood hydrograph into a number of small intervals. The time interval should be so chosen, as not to miss the peak values.

(*ii*) Fix the normal pool level at which the spillway crest is provided, and the level at which the flood enters the reservoir ; the two are generally taken to be the same, as

it is assumed that this worst design flood enters the reservoir only after the reservoir is full up to the normal pool level.

(*iii*) Work out the spillway and the outlet discharge rating curves, if not given.

(*iv*) Work out the elevation-capacity curve for the reservoir from the elevation-area curve, if the former is not given, using cone formula, *i.e.* $V = \Sigma \frac{h}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$, where h is the contour interval. (18.12)

(*v*) Start with the first interval and compute the total inflow during the interval by multiplying the average inflow rate at the beginning and the end of the interval, with the period of the interval.

$$\therefore \qquad I_{(V)} = \frac{I_1 + I_2}{2} (\Delta t)$$

where I_1 = Inflow rate at the start of the interval
I_2 = Inflow rate at the end of the interval
Δt = Duration of the interval
$I_{(V)}$ = Total inflow volume during the interval.

(*vi*) The reservoir level at the start of the flood (*i.e.* start of first interval) is known. Assume a trial value for the reservoir level at the end of the interval.

(*vii*) compute the total outflow during the interval

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot \Delta t$$

where O_1 = Outflow rate at the start of the interval, corresponding to the given reservoir level.
O_2 = Outflow rate at the end of the interval, corresponding to the assumed reservoir level.
Δt = Duration of the interval
$O_{(V)}$ = Total outflow volume during the interval.

(*viii*) Using the elevation-storage curve for the reservoir, determine the storage S_1 and S_2 at the beginning and the end of the interval, corresponding to the known and the assumed reservoir levels, respectively. Their difference $S_2 - S_1 = \Delta S$, represent the amount of flood stored in the reservoir during the interval.

(*ix*) Add the volume of outflow $O_{(V)}$ obtained in step (*vii*) to the values of ΔS obtained in step (*viii*), and compare it with the inflow volume $I_{(V)}$, calculated in step (*v*). The two values must be equal (*i.e.* $I_{(V)} = O_{(V)} + \Delta S$). If this is so, the assumed reservoir level is correct, otherwise, change it and repeat the procedure till this coincidence is obtained.

(*x*) All the above steps should be repeated for other time intervals, till the entire flood is routed or still further, till the reservoir level returns to pre-flood pool level.

(*xi*) Outflow ordinates are plotted so as to obtain the outflow hydrograph. The point at which it crosses the inflow hydrograph gives the peak outflow rate. From this time, the rate of outflow begins to fall due to decrease in the inflow rate.

(*xii*) The time lag between the two peaks is evaluated as to give the time lag.

An example has been solved to make the procedure very clear.

Example 18.13. *The inflow flood discharges for a possible worst flood are tabulated in Table 18.22 at suitable intervals starting from 0.00 hours on august 20, 1975.*

Table 18.22

Time from start in hr	0	6	12	18	24	30	36	42	48	51	60	66	78	90	102	114
Discharge in cumecs	0	50	280	610	1290	1900	2130	1900	1600	1440	1060	780	500	370	220	130

This flood approaches a reservoir with an uncontrolled spillway, the crest of which is kept at RL 140.0 m. Determine the maximum reservoir level and the hydrograph of the routed flood. Values of reservoir capacity (above spillway crest) and outflow discharge at various elevations are tabulated in Tables 18.23 and 18.24 respectively.

Table 18.23

Elevation in metres	*140.0*	*141.0*	*142.0*	*143.0*	*144.0*	*145.0*	*146.0*
Reservoir storage with spillway crest as datum in million cubic metres (m.c.m.)	*0.0*	*15.0*	*35.0*	*60.0*	*95.0*	*140.0*	*240.0*

Table 18.24

Elevation	*Outflow discharge in cumecs*
140.0	*0*
141.0	*170*
142.0	*482*
143.0	*883*
144.0	*1,360*
145.0	*1,905*
146.0	*2,500*

Solution. The elevation storage curve and the elevation outflow curve are plotted with the help of Tables 18.23 and 18.24, as shown in Fig. 18.20 and 18.21 respectively. The hydrograph of the given flood is plotted in Fig. 18.22. Flood routing is carried out by hit and trial method as shown in Table 18.25 and as explained earlier. This table is otherwise self-explanatory.

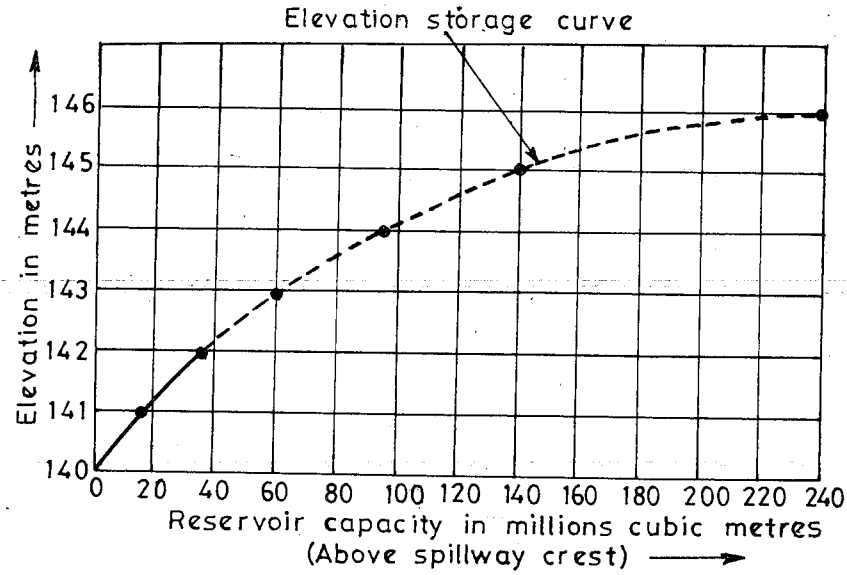


Fig. 18.20. Elevation Storage curve for example 18.13.

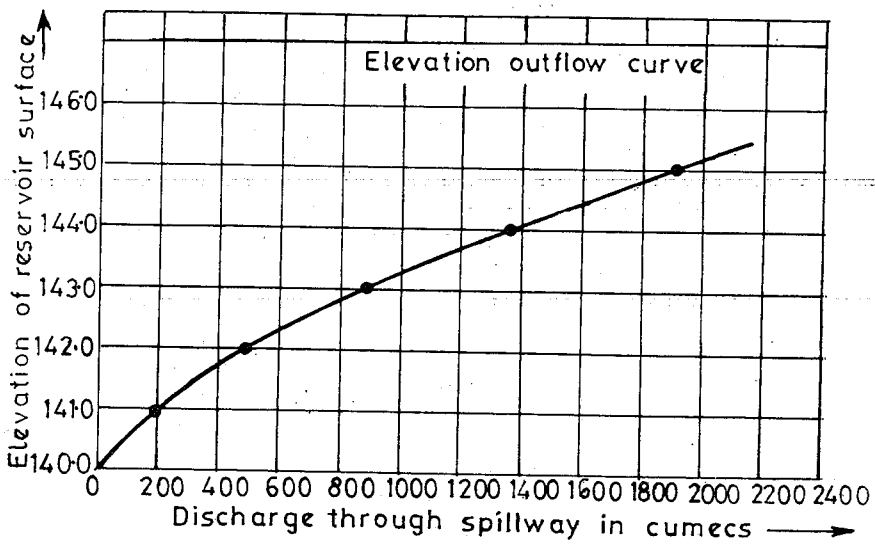


Fig. 18.21. Elevation-Outflow curve for example 18.13.

The outflow hydrograph is plotted from col. (7) of table 18.25, as shown in Fig. 18.22 by the dotted curve. The point at which it intersects the inflow hydrograph, represents the peak of the routed flood.

The peak of outflow flood works out to be **1458 cumecs,** while the peak of the inflow flood was 2,130 cumecs. The time lag is found to be **15 hours** (from Fig. 18.22). The maximum reservoir level is found from Table 18.25, to be **144.18 metres. Ans.**

Table 18.25

Time from start		Time interval (Δt)	Reservoir elevation at the beginning of interval	Inflow at the start of interval (I_1)	Inflow at the end of interval (I_2)	Volume of inflow during the interval $I(V) = (I_1 + I_2)/2$	Outflow at the start of interval (O_1)	Trial reservoir elevation at the end of interval	Outflow at the end of interval (O_2)	Mean outflow volume $O(V) = [(O_1 + O_2)/2]\,\Delta t$	Storage capacity at start of interval (S_1)	Storage capacity at the end of interval (S_2)	Change in storage during the interval $(\Delta S) = (S_2 - S_1)$	Outflow volume + change in storage; i.e. $O_{(V)} + \Delta S$	Compare $O_{(V)} + \Delta S$ with $I_{(V)}$, i.e. col. (14) with col. (6) and Remarks
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
Hrs.		Hrs.	m	I_1 m^3/s	I_2 m^3/s	I $M.m^3$	O_1 m^3/s	m	O_2 m^3/s	$O_{(v)}$ $M.m^3$	S_1 reckoned over spillway crest $M.m^3$	S_2 M.m3	$(S_2 - S_1)$ *i.e.* (ΔS) $M.m^3$	$O_{(V)} + \Delta S$ $M.m^3$	Whether $(O_{(V)} + \Delta S)$ is large or small as compared to $I_{(V)}$
0—6	6		140.0	0.0	50	0.54	0.0	140.1	5.67	0.06	0.0	1.2	1.2	1.26	Large and hence change the trial value
								140.03	0.88	0.01		0.525	0.525	0.535	O.K.

...contd

Table 18.25 (Contd.)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
6-12	6	140.03	50	280	3.56	0.88	140.3	28.7	0.31	0.525	4.0	3.475	3.785	large
							140.25	21.2	0.23	"	3.85	3.325	3.555	O.K.
12-18	6	140.25	280	610	9.62	21.2	140.5	60.0	0.88	3.85	8.0	4.15	5.03	small
							141.0	170.0	2.06	"	15.0	11.15	13.21	large
							140.8	121.6	1.54	"	13.0	9.15	10.69	large
							140.75	110.2	1.42	"	12.05	8.20	9.62	O.K.
18-24	6	140.75	610	1290	20.5	110.2	141.5	312.2	4.57	12.05	26.0	13.95	18.52	small
							141.6	343.0	4.89	"	27.7	15.65	20.54	O.K.
24-30	6	141.06	1290	1900	34.4	343.0	142.5	672	10.95	27.7	47.0	19.3	30.25	small
							142.6	713	11.4	"	50.0	22.3	33.7	small
							142.63	725	11.52	"	50.5	22.8	34.32	O.K.
30-36	6	142.63	1900	2130	43.6	725	143.5	1113	19.85	50.5	76.0	25.5	45.35	large
							143.4	1065	19.23	"	73.0	22.5	41.73	small
							143.45	1087	19.55	"	74.5	24.0	43.55	O.K.
36-42	6	143.45	2130	1900	43.6	1087	144.0	1360	26.44	74.5	95.0	20.5	46.94	large
							143.95	1335	26.16	"	92.0	17.5	43.65	O.K.
42-48	6	143.95	1900	1600	37.8	1335	144.5	1620	13.9	92.0	115.0	23.0	64.9	large
							144.2	1461	30.2	"	102.0	10.0	40.2	large
							144.15	1435	29.9	"	100.0	8.0	37.9	O.K.

...contd

Table 18.25 (Contd.)

(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*	*(14)*	(15)
48-51	3	144.15	1600	1440	16.4	1435	114.18	1458*	15.5	100.0	101.0	1.0	16.5	O.K.
51-60	9	144.18*	1440	1060	40.5	1458*	144.10	1410	46.5	101.0	98.0	–3.0	43.5	large
							144.05	1385	46.1	"	95.5	–5.0	41.1	O.K.
60-66	6	144.05	1060	780	19.9	1385	143.8	1258	28.5	95.5	87.0	–8.5	20.0	O.K.
66-78	12	143.80	780	500	27.8	1258	142.2	974	48.2	87.0	67.0	–20.0	28.2	O.K.
78-90	12	143.20	500	370	18.8	974	142.60	713	35.4	67.0	50.0	–17.0	18.4	O.K.
90-102	12	142.60	370	220	12.7	713	142.0	482	25.8	50.0	35.0	–15.0	10.8	small
							142.08	510	26.5		36.2	–13.8	12.7	O.K.

*Results : Peak rate of outflow = 1,458 cumecs
Maximum reservoir level = 144.18 metres
Time lag = 15 hours

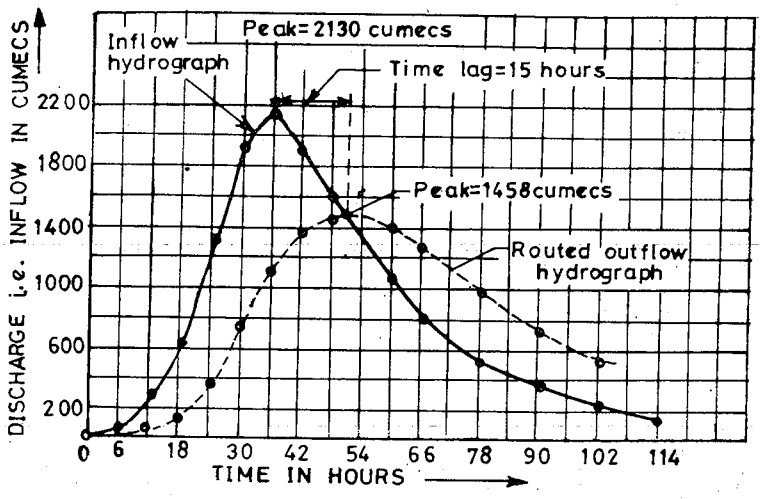


Fig. 18.22. Hydrograph of the given flood and that of the routed flood for example 18.10.

Example 18.14. *The hydrograph of inflow to a reservoir is given in the table below.*

Time(days)	*0*	*2*	*4*	*6*	*8*	*10*	*12*	*14*	*16*	*18*	*20*
Flow (m^3/s)	*60*	*115*	*425*	*550*	*440*	*320*	*260*	*200*	*150*	*110*	*60*

The reservoir is full at the start of the flood inflow. The storage S of reservoir above the spillway crest, is given in million cubic metres by : $S = 8.64\,h$, where h is the head in metres above the crest. The discharge over the spillway is given in cumecs by $Q = 60\,h$. Find the head over the spillway crest at the end of 8th day of the flood.

Solution. Let h_1 be the head over the spillway crest at the end of 2 days ; h_2 at the end of 4 days ; h_3 at the end of 6 days ; and h_4 at the end of 8 days. Now we will consider the position, interval by interval.

(*i*) **0—2 day interval**

Inflow $I_1 = 60$ cumecs.

$I_2 = 115$ cumecs.

$$\therefore \quad I_{(V)} = \left(\frac{I_1 + I_2}{2}\right) t = \left(\frac{60 + 115}{2}\right) 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 15.12\ \text{M.m}^3.$$

Outflow $O_1 = 0$

$O_2 = 60\,h_1$

$$O_{(V)} = \frac{O_1 + O_2}{2} \cdot t = \left(\frac{60h_1}{2}\right) \times 2 \times (24 \times 60 \times 60)\ \text{m}^3 = 5.184\,h_1\ \text{M.m}^3.$$

Storage $S_1 = 0$

$S_2 = 8.64\,h_1\ \text{M.m}^3$

$\therefore \quad \Delta S = 8.64\,h_1$

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Now using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$15.12 = 5.184\, h_1 + 8.64\, h_1 \quad \text{or} \quad 15.12 = 13.824\, h_1$$

$$\therefore \quad h_1 = \frac{15.12}{13.824} = \mathbf{1.093\ m.}$$

(*ii*) **2—4 day interval**

Inflow $I_1 = 115$ cumecs ; $I_2 = 425$ cumecs

$$I_{(V)} = \left(\frac{115 + 425}{2}\right) 2 \times 86400\ \text{m}^3 = 46.656\ \text{Mm}^3.$$

Outflow $O_1 = 60\, h_1$; $O_2 = 60\, h_2$

$$O_{(V)} = \left(\frac{60h_1 + 60h_2}{2}\right) 2 \times (86400)\ \text{m}^3 = (h_1 + h_2) \times 5.184\ \text{Mm}^3.$$

Storage $S_1 = 8.64\, h_1$; $S_2 = 8.64\, h_2$

$$\Delta S = 8.64\, (h_2 - h_1)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$46.656 = (h_1 + h_2)\ 5.184 + 8.64\, (h_2 - h_1)$$

Using $h_1 = 1.093$ m, we get

$$46.656 = 5.666 + \mathbf{5.184\, h_2} + \mathbf{8.64\, h_2} - 9.443$$

or $$50.433 = 13.824\, h_2 \quad \text{or} \quad \mathbf{h_2 = 3.648\ m.}$$

(*iii*) **4—6 day interval**

Inflow $I_1 = 425$ cumecs ; $I_2 = 550$ cumecs

$$\therefore \quad I_{(V)} = \frac{425 + 550}{2} \times 2 \times 86400\ \text{m}^3 = 84.24\ \text{M.m}^3.$$

Outflow $O_1 = 60h_2$; $O_2 = 60h_3$

$$O_{(V)} = \frac{60h_2 + 60h_3}{2} \times 2 \times 86400\ \text{m}^3 = (h_2 + h_3)\ 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64h_2$; $S_2 = 8.64h_3$

$$\Delta S = 8.64\, (h_3 - h_2)$$

Using $I_{(V)} = O_{(V)} + \Delta S$, we get

$$84.24 = (h_2 + h_3)\ 5.184 + 8.64\, (h_3 - h_2)$$

Using $h_2 = 3.648$ m in this eqn., we get

$$84.24 = 18.911 + \mathbf{5.184\ h_3} + \mathbf{8.64\, h_3} - 31.518$$

$$\therefore \quad 96.848 = 13.824\, h_3 \quad \therefore \quad \mathbf{h_3 = 7.00\ m.}$$

(*iv*) **6–8 day interval.**

Inflow $I_1 = 550$ cumecs ; $I_2 = 440$ cumecs

$$I_{(V)} = \left(\frac{550 + 440}{2}\right) 2\ (86400)\ \text{M.m}^3 = 85.536\ \text{M.m}^3.$$

Outflow $O_1 = 60\, h_3$; $O_2 = 60\, h_4$

$$O_{(V)} = \frac{60h_3 + 60h_4}{2} \times 2 \times 86400\ \text{m}^3 = (h_3 + h_4)\ 5.184\ \text{M.m}^3.$$

Storage $S_1 = 8.64\, h_3$; $S_2 = 8.64\, h_4$

$$\therefore \quad \Delta S = S_2 - S_1 = 8.64\, (h_4 - h_3)$$

Using $h_3 = 7.00$ m in this eqn., we get

$$85.536 = 36.288 + \mathbf{5.184\, h_4} + \mathbf{8.64\, h_4} - 60.48$$

or $109.728 = 13.824\, h_4$ or $\mathbf{h_4 = 7.94\ m.}$

Hence, the head over the spillway crest at the end of the 8th day = 7.94 m. **Ans.**

Example 18.15. *A small reservoir has an area of 5000 hectares* ($50 \times 10^6\ m^2$) *at spillway crest level. Banks are essentially vertical above the spillway crest level. The spillway is 50 m long and has a coefficient C of 2.2 in equation* $q = CH^{3/2}$. *The inflow to the reservoir is given in the table below :*

Time from start (hrs.)	*0*	*6*	*12*	*24*	*30*	*36*	*42*	*48*	*54*
Inflow (m^3/s)	*40*	*340*	*722*	*320*	*192*	*118*	*80*	*56*	*40*

Compute the maximum outflow discharge over the spillway; and the reservoir level to be expected if the reservoir level was at the spillway crest at the start.

(Bhopal University, 1980)

Solution. As done in example 18.11, let us assume that the head over the spillway crest at the end of the various intervals be h_1, h_2, h_3, h_4 metres. Then

For interval 0—6 hr.

Inflow $I_1 = 40$ cumecs ; $I_2 = 340$ cumecs

$$I_{(V)} = \left(\frac{40 + 340}{2}\right)(6 \times 60 \times 60)\ m^3 = 4.104\ M.m^3.$$

Outflow $O_1 = 0$

$$O_2 = 2.2 \times 50 \times h_1^{3/2} = 110\, h_1^{3/2} \qquad [\because Q \text{ over spillway} = C.L.H.^{3/2}]$$

$$O_{(V)} = \left(\frac{O_1 + O_2}{2}\right) \cdot t = \left(\frac{0 + 110 h_1^{3/2}}{2}\right)(6 \times 60 \times 60)\ m^3 = 1.19\, h_1^{3/2}\ M.m^3.$$

Storage $S_1 = 0$

$$S_2 = (50 \times 10^6)\, h_1\ m^3. \qquad [\because \text{Storage above the spillway is rectangular} = \text{Area of the reservoir} \times \text{ht. of water}]$$

$$\therefore \quad \Delta S = 50\, h_1\ M.m^3.$$

Now, using $I_{(V)} = O_{(V)} + \Delta S$, we have

$$4.104 = 1.19\, h_1^{3/2} + 50 h_1 \qquad \text{or} \qquad h_1^{3/2} + 42 h_1 - 3.45 = 0$$

Solving by hit and trial, we get

$$\mathbf{h_1 = 0.082\ m.}$$

For interval 6–12 hr.

$$I_{(V)} = \left(\frac{340 + 722}{2}\right)(6 \times 60 \times 60)\ m^3 = 11.47\ M.m^3$$

$$O_{(V)} = \frac{110\, h_1^{3/2} + 110\, h_2^{3/2}}{2} \cdot (6 \times 60 \times 60)\ m^3 = (h_1^{3/2} + h_2^{3/2})\ 1.19\ M.m^3$$

$$\Delta S = 50\,(h_2 - h_1)\ M.m^3$$

$$\therefore \quad 11.47 = 1.19\,(h_1^{3/2} + h_2^{3/2}) + 50(h_2 - h_1)$$

Using $h_1 = 0.082$, we have

$$9.65 = 0.02 + h_2^{3/2} + 42h_2 - 3.44 \quad \text{or} \quad 13.07 = h_2^{3/2} + 42h_2,$$

or $h_2^{3/2} + 42h_2 - 13.07 = 0$ $\therefore$ $\mathbf{h_2 = 0.31\ m.}$

For interval 12—24 hr.

Inflow $I_{(V)} = \left(\frac{722 + 320}{2}\right)(12 \times 60 \times 60)\ \text{m}^3 = 22.51\ \text{M.m}^3$

Outflow $O_1 = 110\ h_2^{3/2}$; $O_2 = 110\ h_3^{3/2}$

$$O_{(V)} = \frac{110\ (h_2^{3/2} + h_3^{3/2})}{2} \times (12 \times 60 \times 60)\ \text{m}^3 = 2.38\ (h_2^{3/2} + h_3^{3/2})\ \text{M.m}^3$$

Storage $S_1 = 50\ h_2$ $S_2 = 50\ h_3$

$$\Delta S = 50\ (h_3 - h_2)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$\therefore$ $22.51 = 2.38\ (h_2^{3/2} + h_3^{3/2}) + 50\ (h_3 - h_2)$

Using $h_2 = 0.31$ m, we have

$$9.46 = 0.17 + h_3^{3/2} + 12.2\ h_3 - 3.78$$

$$13.07 = h_3^{3/2} + 12.2\ h_3.$$

or $h_3^{3/2} + 12.2\ h_3 - 13.07 = 0$ $\therefore$ $\mathbf{h_3 = 0.99\ m.}$

For interval 24—30 hr.

Inflow $I_{(V)} = \left(\frac{320 + 192}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 5.53\ \text{M.m}^3$

Outflow $O_{(V)} = \frac{110\ (h_3^{3/2} + h_4^{3/2})}{2}\ (6 \times 60 \times 60) = 1.19\ (h_3^{3/2} + h_4^{3/2})$

$$\Delta S = 50\ (h_4 - h_3)$$

$$I_{(V)} = O_{(V)} + \Delta S.$$

$\therefore$ $5.53 = 1.19\ (h_3^{3/2} + h_4^{3/2}) + 50\ (h_4 - h_3)$

or $4.65 = (h_3^{3/2} + h_4^{3/2}) + 9.04\ (h_4 - h_3)$

Using $h_3 = 0.99$. we have

$$4.65 = 0.99 + h_4^{3/2} + 9.04\ h_4 - 8.95$$

or $h_4^{3/2} + 9.04 h_4 - 12.61 = 0$ $\therefore$ $\mathbf{h_4 = 1.24\ m.}$

For interval 30—36 hr.

$$I_{(V)} = \left(\frac{192 + 118}{2}\right)(6 \times 60 \times 60)\ \text{m}^3 = 3.35\ \text{M.m}^3.$$

$$O_{(V)} = 1.19\ (h_4^{3/2} + h_5^{3/2})$$

$$\Delta S = 50\ (h_5 - h_4)$$

$\therefore$ $3.35 = 1.19\ (h_4^{3/2} + h_5^{3/2}) + 50\ (h_5 - h_4)$

or $2.81 = (h_4^{3/2} + h_5^{3/2}) + 42\,(h_5 - h_4)$

Using $h_4 = 1.24$ m, we get

$$2.81 = 1.38 + h_5^{3/2} + 42\,h_5 - 52.08$$

or $h_5^{3/2} + 42h_5 - 50.65 = 0 \qquad \therefore \qquad h_5 = 1.17$ m

Since $h_5 < h_4$, it means that the head has started reducing, reaching its max. value of **1.24 m.**

$\therefore$ max. outflow discharge over the spillway

$$= 110\,(1.24)^{3/2} \text{ cumecs.} = 151.8 \text{ cumecs.} \quad \textbf{Ans.}$$

and, the max. reservoir level will be 1.24 m higher than the spillway crest level, which cannot be worked out as the spillway crest level is not given. **Ans.**

RESERVOIR REGULATION

18.12. Rule Curves and Operating Tables for Reservoirs

Multipurpose reservoirs need to be operated and regulated efficiently with a high degree of intelligence, intuition, and experience, in order to ensure that *they are neither left partially empty by the end of the rainy season, nor they are found full at the time of arrival of a series of peak floods, leading to heavy releases, causing floods in the downstream valley.*

Reservoir regulation committees, consisting of experts are, therefore, generally constituted to ensure issue of proper and timely directives to the staff operating the gated openings of the reservoir, to avoid any bad and inefficient operations.

Guiding tables and curves, called **rule curves** or **guide curves,** are drawn in advance, and kept ready for use for the efficient regulation of the reservoir waters, with time. Such guiding curves are normally required only for the flood season, because for the rest of the year, the reservoir will only discharge water for irrigation and hydel needs. Sometimes, **reservoir regulation manuals** are also prepared and made available to the officers, dealing with the operation of the gates of the reservoir. Such manuals provide guidelines for gate operations, and for the overall maintenance and upkeep of the dam and the reservoir.

A rule curve, either in tabular or graphical form, as framed by the experts on the basis of the past data, broadly reflects the maximum reservoir levels to be achieved by the different dates of the rainy season. *Such a curve, therefore, reflects the vacant space to be left in the reservoir on different dates or weeks of the rainy season.* Such guides may have to be revised from time to time, based on their performance during their actual use.

A typical set of such rule curves (tabular form) being followed for the operation of *Hirakud dam reservoir** are shown in table 18.26 (*a*) and (*b*).

* Hirakud dam is constructed on Mahanadi river is Orissa State of India. It is 3 mile long composite dam, with max. height of 200 ft. Spillway capacity at FRL of 630′ is 14.83 lakh cusecs. It intercepts a catchment of 32,200 sq. miles, and the catchment below the dam and up to the head of delta, i.e. Naraj, is about 18,000 sq. miles. There is no flood reserve or conservation reserve as such, and it is a general use reservoir. The river capacity at Naraj is about 9 lakh cusecs.

Table 18.26 (*a*) Rule curve for filling the Hirakud reservoir for conservation

Date of the year	*Permissible reservoir water level (Dead storage RL=590ft)*
11th August (on and upto)	600′
21st August	605′
1st September	617′
11th September	623′
21st September	627′
1st October (on and after)	630′ (F.R.L.)

Table 18.26 (*b*) Rule curve for flood releases (under ordinary circumstances) for Hirakud dam reservoir

Range of levels RL in ft	*Permissible outflow in lakh cusecs*	*Safe depletion rate in lakh cusecs*
600 — 610	2.5	4.0
610 — 615	3.0	
615 — 620	4.0	
620 — 625	4.5	
625 — 628	5.5	
628 — 630	6.5	

Prior to the above rules, Hirakud dam reservoir was following another set of rules, as given in table 18.27, which proved to be inadequate, leading to insufficient storage at the end of flood seasons for some of the years, and heavy discharges from the reservoir synchronising with floods in the lower catchment in some other years.

Table 18.27. Old Rules, specified in the Manual for Operation of Hirakud reservoir, which have now been superceded by rules of tables 18.21 (*a*) and (*b*)

S. No. of Rule	*Rule*
Rule 1	There should be no full impounding till 1st of September in normal years. The entire inflow should be let out ; care being exercised that the combined discharge of reservoir outflow and the runoff from the catchment below the dam does not exceed 10 lakh cusecs at Naraj. Generally, the inflow up to 5 lakh cusecs can be passed without hesitation. When the inflow is more than 5 lakh cusecs, then the hydrometeorological conditions in the lower catchment should be carefully studied and outflow determined.
Rule 2	Safety of dam should be the prime consideration on and at every occasion when floods are absorbed in the reservoir for purposes of moderating and regulating outflow in the reservoir. A level of + 625.0′ should be deemed to be the safe level normally for this purpose.

Another typical rule curve (graphical form), as drawn and used for Maithon dam reservoir, under *D.V.C. System**, is shown in Fig. 18.23. This operation schedule shows that the reservoir, which will be quite depleted by the start of June, will be filled up fully up to *Monsoon storage level* (RL 480′), latest by 3rd week of September, and will be maintained full, till the end of November. The reservoir will be allowed to be depleted only from December onward.

Any flood, which occurs during the last week of September, October and November, will thus, find the reservoir full, and will have to be accommodated within the *'flood reserve', i.e.* between monsoon storage level (RL 480′) and the max. permissible operational level of gates (RL 495′).

Depending upon the intensity of the coming and likely floods after the reservoir is full, the outflows will be regulated and so manipulated that the discharge from the reservoir together with the discharge contributed by the downstream area, does not exceed the safe carrying capacity of the river farther downstream.

As and when a late flood enters the reservoir, the reservoir-gate-operators may adjust the outflow to a lower value in the beginning, but may have to increase the outflow, as the flood waves continue and the weather forecasts for continued rains are received. In this operation, a stage may reach when the outflow may become quite high, causing floods in the down valley.

While manipulating these outflows, chances of synchronisation of the discharge of the downstream uncontrolled catchment will also have to be considered properly.

Say for example, if we release 1 lakh cusecs discharge from the reservoir, and another 1 lakh cusecs gets added to it from the catchment of the downstream 1 km length of the channel, then eventually 2 lakh cusecs will be flowing in the channel farther down the 1 km length. This 2 lakh cusecs may prove quite harmful, as it may exceed the safe carrying capacity of the downstream river (below 1 km).

Another possibility for gate operators could then be to release lesser discharge, say 50,000 cusecs, which with 1 km downstream contribution of 1 lakh cusecs, will make up 1,50,000 cusecs, which may not prove dangerous to farther down. But releasing lesser quantity would cause the reservoir water level to rise rapidly, encroaching the 'flood reserve' quickly, finally leading to a stage, wherein you may have to open the entire gates, passing say 4 lakh cusecs, to avoid over-topping and failure of the dam. This 4 lakh cusecs will eventually destroy the entire downstream area, and may prove worser than what would have happened, if 1 lakh cusecs would have been allowed in the beginning itself.

You can, thus, understand and appreciate as to how important it is, to properly man the outflows from a reservoir. The intelligent and timely manipulations, accompanied by proper rain forecasts, may help us to avoid flooding of the downstream area.

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* D.V.C. (Damodar Valley Corporation) was constituted in India in July, 1948, on the line of T.V.A. (Tennesse Valley Authority) of USA. to plan and execute flood control works on river Damodar and its tributaries, which had created havoc in the year 1943, causing serious flood losses of the order of Rs. 8 crores, and cutting off the most important Calcutta city with the rest of the country for about ten weeks, due to breaches in G.T. Road and Rly link.

Consequently, 4 dams were constructed, including the Maithon on river Damodar, Konar on river Konar, and Panchat and Tallaiya on its tributary Barakar, along with a barrage at Durgapur (situated a few km downstream of the confluence of Damodar and Barakar. Konar and *Talliya* are smaller dams located in the upper reaches, whereas, Maithon and Panchat are bigger dams in lower reaches, bearing most of the flood burden.

Maithon is located about 13 km above the confluence, and 21 km northwest of Asansol and 40 km east of Dhanbad. Like Konar, it is partly earthen and partly of concrete. The primary purpose of the dam is flood control; whereas, *irrigation* and *hydropower* are secondary purposes.

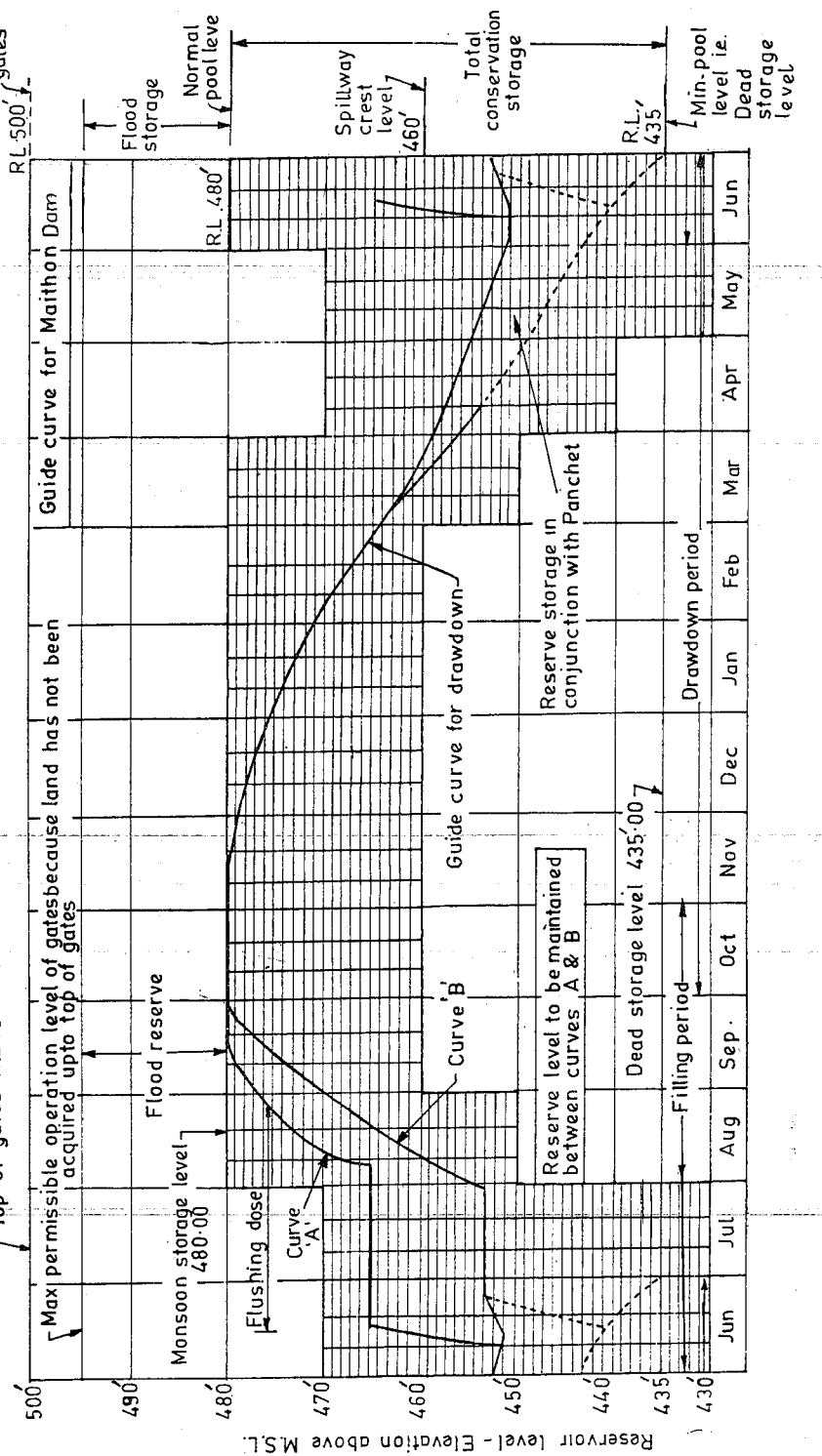


Fig. 18.23. A typical operating guide for Maithon Dam Reservoir.

18.13. Reservoir Sedimentation

Every river carries certain amount of sediment load. The sediment particles try to settle down to the river bottom due to the gravitational force, but may be kept in suspension due to the upward currents in the turbulent flow which may overcome the gravity force. Due to these reasons, the river carries fine sediment in suspension as suspended load, and larger solids along the river bed as bed load. When the silt laiden water reaches a reservoir in the vicinity of a dam, the velocity and the turbulence are considerably reduced. The bigger suspended particles and most of the bed load, therefore, gets deposited in the head reaches of the reservoir. Fine particles may travel some more distance and may finally deposit farther down in the reservoir, as shown in Fig. 18.24. Some very fine particles may remain in suspension for much longer period, and may finally escape from the dam along with the water discharged through the sluiceways, turbines, spillway, etc.

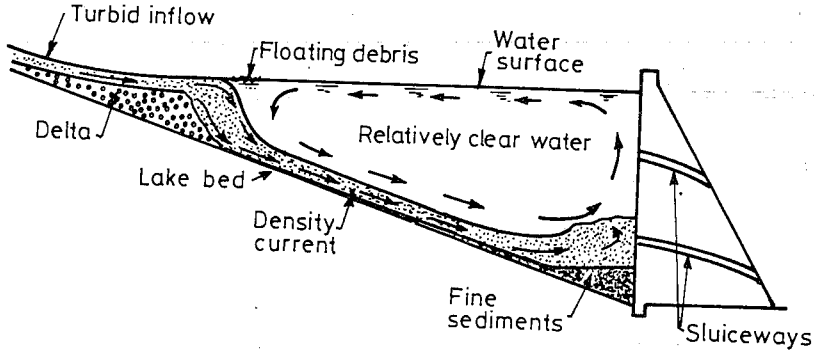


Fig. 18.24. Sediment accumulation in a typical reservoir.

The deposition of sediment in the reservoir is known as 'Reservoir Silting' or 'Reservoir Sedimentation'.

The deposition of the sediment will automatically reduce the water storing capacity of the reservoir, and if this process of deposition continues longer, a stage is likely to reach when the whole reservoir may get silted up and become useless.

Moreover, with the passage of time, the reservoir capacity will go on reducing. Thus, if today at the time of construction, a reservoir can store 10,000 cubic metres of water, tomorrow say after five years, it may be able to store only say 8,000 cubic metres of water. Therefore, in order to see that the capacity does not fall short of requirement ever during the design period, we must take this silting into account. The total volume of silt likely to be deposited during the designed life period of the dam is, therefore, estimated ; and approximately that much of volume is left unused to allow for silting, and is known as **dead storage.** The remainder is known as the *effective storage* or the **live storage.** The dead storage generally varies between 15 to 25% of the total capacity. For example in Bhakra dam, the gross capacity of the dam is 9,344 million cubic metres and the dead storage provided is 2,054 million cubic metres. All the outlets fetching water from the reservoir are provided above the dead storage level.

The importance of this silting can be understood by considering the following example :

Let the total capacity of a reservoir be 30 million cubic metres and the provision of dead storage be 6 million cubic metres. Let the average volume of sediment deposition be 0.15 million cubic metres per year. Then it is evident, that the dead storage will be filled up in $\frac{6}{0.15} = 40$ years, and the total storage in about $\frac{30}{0.15} = 200$ years.

Hence, the usefulness of this reservoir would start reducing after 40 years, and after 200 years it would be nothing but a collection of sand and sediment with no water in it, provided the siltation rate remains constant at 0.15 M.m^3/yr.

18.13.1. Density Currents. In a reservoir, the coarser sediment settles down along the bottom of the reservoir, as the muddy flow approaches the reservoir; while the finer sediment usually remains in suspension, and moves in a separate layer than the clear reservoir water, as shown in Fig. 18.24. This layer of water, containing the fine sediment, moves below the upper clearer reservoir water, as a *density current*, since its density is slightly more than the density of the main body of the reservoir water. *Because of their density difference, the water of the density current does not mix easily with the reservoir water, and maintains its identity for a considerable time.* The density current can thus be removed through the dam sluiceways, if they are located properly and at the levels of the density current. A lot of sediment load can, thus, be passed out of the reservoir, if it is possible to locate the dam outlets and sluiceways in such a fashion, as to vent out the density currents. Trap efficiency of reservoirs may thus be decreased by about 2 to 10%, if it is possible to vent such density currents through the outlets and sluiceways of the dams.

18.13.2. Trap Efficiency. Now, we introduce another very important term called Trap efficiency. Trap efficiency is defined as the percentage of the sediment deposited in the reservoir even inspite of taking precautions and measures to control its deposition.

Therefore, Trap Efficiency (η)

$$= \frac{\text{Total sediment deposited in the reservoir}}{\text{Total sediment flowing in the river}} \qquad ...(18.13)$$

Most of the reservoirs trap 95 to 100% of the sediment load flowing into them. Even if various feasible silt control measures are adopted, it has not been possible to reduce this trap efficiency below 90% or so.

18.13.3. Capacity Inflow Ratio. The ratio of the reservoir capacity to the total inflow of water in it, is known as the capacity-inflow ratio. It is a very important factor, because the trap efficiency (η) has been found to be a function of capacity-inflow ratio *i.e.*

$$\eta = f\left(\frac{\text{Capacity}}{\text{Inflow}}\right) \qquad ...(18.14)$$

The graph obtained for the existing reservoirs between trap efficiency and log of $\left(\frac{\text{Capacity}}{\text{Inflow}}\right)$ has been found to be of the type shown in Fig. 18.25.

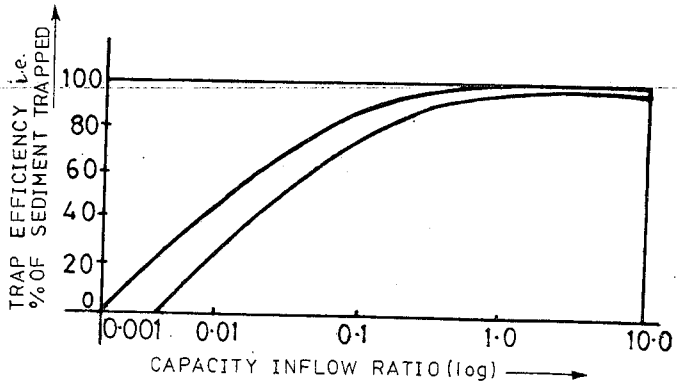


Fig. 18.25.

It is evident from the above curve, that if capacity reduces (with constant inflow), trap efficiency reduces, and hence, lesser sediment is trapped. Therefore, the silting rate in the reservoir shall be more in the beginning, and as its capacity reduces due to silting, the silting rate will reduce. Hence, the complete reservoir-silting may take longer period.

It can also be concluded that for small reservoirs (having small capacity) on large rivers (having large inflow rates), the trap efficiency is extremely low, because the capacity inflow ratio is very small. Such reservoirs silt very little and most of their sediment is passed downstream. On the other hand, large reservoirs on smaller rivers shall silt tremendously and almost complete deposition of sediment may take place.

18.13.4. Silting of Power Reservoirs. In case of reservoirs constructed solely for the purpose of power generation, the silting is comparatively less important. This is because of the fact, that for the proper and efficient functioning of a power reservoir, only a certain minimum head is necessary. This head remains available even after some silt gets deposited. So, only if sufficient water required for power generation remains available, the reservoir's efficiency remains unaffected by silting. But due to silted water, the abrasion of the blades of the turbines may occur very soon, and power production may be stopped over a considerable length of time.

18.13.5. Silting Control in Reservoirs. In order to increase the life of a reservoir, it is necessary to control the deposition of sediment. Various measures are undertaken in order to achieve this aim. The various methods which are adopted can be divided into two parts :

(1) Pre-constructing measures ; and (2) Post-constructing measures.

These measures are discussed below :

(1) **Pre-constructing measure.** They are those measures which are adopted before and during the execution of the project. They are innumerated below :

(*a*) *Selection of Dam Site.* The silting depends upon the amount of erosion from the catchment. If the catchment is less erodible, the silting will be less. Hence, the silting can be reduced by choosing the reservoir site in such a way as to exclude the run off from the easily erodible catchment.

(*b*) *Construction of the Dam in Stages.* The design capacity plays an important role in the silting of a reservoir. When the storage capacity is much less than the average annual runoff entering the reservoir, a large amount of water will get out of the reservoir, thereby, reducing the silting rate compared to what it would have been if the entire water would have been stored. Therefore, the life of a reservoir can be prolonged by constructing the dam in stages. In other words, first of all, the dam should be built lower, and raised subsequently when some of its capacity gets silted up.

(*c*) *Construction of Check Dams.* The sediment inflow can be controlled by building check dams across the river streams contributing major sediment load. These are smaller dams and trap large amounts of coarser sediments. They are quite expensive.

(*d*) *Vegetation Screens.* This is based on the principle that vegetations trap large amounts of sediment. The vegetation growth is, therefore, promoted at the entrance of the reservoir as well as in the catchment. These vegetative covers, through which flood waters have to pass before entering the reservoirs, are known as vegetation screens, and provide a cheap and a good method of silt control.

(*e*) *Construction of Under-sluices in the Dam.* The dam is provided with openings in its base, so as to remove the more silted water on the downstream side.

The sediment concentration will be more at some levels than at others. Therefore, sluices are located at the levels of higher sediment concentration. The method in itself, is not sufficient because the water digs out a channel behind the sluice for movement and leaves most of the sediment undisturbed. Therefore, this is simultaneously supplemented with mechanical loosening and scouring of the neighbouring sediment in order

to increase its effectiveness. But to provide large sluices near the bottom of the dam, is again a structural problem. The use of this method is, therefore, limited.

(2) **Post-constructing Measures.** These measures are undertaken during the operation of the project. They are given below:

(1) *Removal of Post Flood Water.* The sediment content increases just after the floods ; therefore, attempts are generally made not to collect this water. Hence, the efforts should be made to remove the water entering the reservoir at this time.

(2) *Mechanical Stirring of the Sediment.* The deposited sediment is scoured and disturbed by mechanical means, so as to keep it in a moving state, and thus, help in pushing it towards the sluices.

(3) *Erosion Control and Soil Conservation.* This includes all those general methods which are adopted to reduce erosion of soil and to make it more and more stable. This method is the most effective method for controlling siltation, because when the soil erosion is reduced, the sedimentation problem is reduced automatically. But the methods of treating the catchment in order to minimise erosion are very costly. It has been estimated that the investment required for treating 16% of the Indian catchment area is Rs. 1,000 crores. In India, only 1.5% of the catchment area has been treated to minimise silting.

Example 18.16. *The following information is available regarding the relationship between trap efficiency and capacity inflow ratio.*

Capacity inflow ratio	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1.0*
Trap efficiency percent	*87*	*93*	*95*	*95.5*	*96*	*96.5*	*97*	*97*	*97*	*97.5*

Find the probable life of the reservoir with an initial reservoir capacity of 30 million cubic metres, if the average annual flood inflow is 60 million cubic metres and the average annual sediment inflow is 2,00,000 tonnes. Assume a specific weight of the sediment equal to 1.2 gm per c.c. The usual life of the reservoir will terminate when 80% of its initial capacity is filled with sediment.

Solution.

Average annual sediment inflow = 2,00,000 tonnes

$$= 2 \times 10^5 \text{ tonnes} = 2 \times 10^{11} \text{ gm}$$

Volume of average annual sediment inflow

$$= \frac{2 \times 10^{11}}{1.2} \text{ c.c.} = \frac{2 \times 10^{11}}{1.2 \times 10^6} \text{ m}^3$$

$$= \frac{0.2}{1.2} \times 10^6 \text{ cubic metres} = \frac{1}{6} \text{ million cubic metre} = \frac{1}{6} \text{ M.m}^3$$

Initial Reservoir Capacity $= 30 \text{ M.m}^3$

Annual flood inflow $= 60 \text{ M.m}^3$

Let us assume that 20% of the capacity, *i.e.* 6 M.m^3 is filled up in the first interval.

Capacity inflow ratio at the start of the interval $= \frac{30}{60} = 0.5$

Trap efficiency at the start of the interval = 0.96.

Capacity inflow ratio at the end of the interval $= \frac{24}{60} = 0.4$

Trap efficiency at the end of interval = 0.955

Average trap efficiency during the interval $= \frac{0.96 + 0.955}{2} = \mathbf{0.9575.}$

Volume of sediment deposited annually till the 20% capacity is filled

$$= \frac{1}{6} \times 0.9575 \text{ M.m}^3$$

$\therefore$ No. of years during which 20% of the capacity, *i.e.* 6 M.m^3 shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9575} \text{ years} = \frac{36}{0.9575} = \mathbf{37.6\ years}$$

Similarly, in the 2nd interval

Capacity inflow ratio at the start $= \frac{24}{60} = 0.4$

Capacity inflow ratio at the end $= \frac{18}{60} = 0.3$

Trap efficiency at the start $= 0.955$
Trap efficiency at the end $= 0.95$
Average trap efficiency $= \mathbf{0.9525.}$

$\therefore$ No. of years during which the next 20% of capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9525} = \frac{36}{0.9525} = \mathbf{37.8\ years.}$$

Similarly, in the 3rd interval

Capacity inflow ratio at start $= \frac{18}{60} = 0.3$

Capacity inflow ratio at the end $= \frac{12}{60} = 0.2$

Trap efficiency at the start $= 0.95$
Trap efficiency at the end $= 0.93$
Average trap efficiency during the interval $= 0.94$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.94} = \frac{36}{0.94} = \mathbf{38.3\ years.}$$

Similarly, in the 4th interval

Capacity inflow ratio at the start $= 12/60 = 0.2$
Capacity inflow ratio at the end $= 6/60 = 0.1$
Trap efficiency at the start $= 0.93$
Trap efficiency at the end $= 0.87$

Average trap efficiency during the interval $= 0.90$

No. of years during which the next 20% of the capacity shall be filled up

$$= \frac{6}{\frac{1}{6} \times 0.9} = \frac{36}{0.9} = \mathbf{40\ years.}$$

Total probable life till 80% capacity gets filled up

$$= 37.6 + 37.8 + 38.3 + 40.0 = \mathbf{153.7\ years.} \quad \textbf{Ans.}$$

The above calculations of dividing the entire capacity into intervals (20% each in the above case) can also be carried out in a tabular form, as shown below in Table 18.28.

Table 18.28

Capacity		Capacity/inflow $= \frac{\text{Col. (2)}}{60\ \text{Mcum}}$	Trap efficiency η	Av. Trap eff. η_{av} during the interval	Sediment trapped per year; Col. (5) × Av. annual sediment inflow = Col. $5 \times \frac{1}{6}$ M.cum.	Years reqd. to fill up 20% capacity (6 Mcum.) $= \frac{6}{\text{Col. (6)}}$ in years
%	Capacity Vol. in M. cum.					
(1)	(2)	(3)	(4)	(5)	(6)	(7)
100	30	0.5	0.96			
				0.9575	0.1596	36.6
80	24	0.4	0.955			
				0.9525	0.1588	37.8
60	18	0.3	0.95			
				0.94	0.1566	38.3
40	12	0.2	0.93			
				0.90	0.15	40.0
20		0.1	0.8			
						$\Sigma = 153.7$ yrs

Example 18.17. *A proposed reservoir has a capacity of 500 ha-m. The catchment area is 125 km^2, and the annual streamflow averages 12 cm of runoff. If the annual sediment production is 0.03 ha.m/km^2, what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation? The relationship between trap efficiency η (%) and capacity inflow ratio C/I, is as under :*

C/I	0.01	0.02	0.04	0.06	0.08	0.1	0.2	0.3	0.5	0.7
η%	43	60	74	80	84	87	93	95	96	97

(U.P.S.C., Civil Services, 1987)

Solution. Av. annual streamflow = 12 cm of runoff

Area of catchment $= 125\ \text{km}^2 = 125 \times 10^6\ \text{m}^2$

$\therefore$ Annual flood inflow $= (125 \times 10^6) \cdot \frac{12}{100}\ \text{m}^3$

$= 15 \times 10^6\ \text{m}^3 = \mathbf{15\ M.m^3\ (Mcum)}.$

Annual sediment inflow = 0.03 ha-m/km^2 of the catchment

$= 0.03 \times 125$ ha–m $= 0.03 \times 125 \times 10^4\ \text{m}^3 = 3.75 \times 10^4\ \text{m}^3$

$= \frac{3.75}{100} \times 10^6\ \text{m}^3 = \mathbf{0.0375\ M\text{–}m^3\ (Mcum)}.$

It means that 0.0375 Mcum of sediment flows every year into the dam/reservoir site, but the quantum of this, which is trapped in the reservoir, depends upon the average trap efficiency (η) during that year, and this trap efficiency, in turn, depends upon the capacity/inflow ratio.

In the question, the total capacity to be filled up by sediment is 10% of the initial reservoir capacity,

i.e. $10\% \times 5\ \text{Mcum} = 0.5\ \text{Mcum}.$

Now, we have to calculate the time during which this 0.5 Mcum of sediment will get deposited in the reservoir, as follows :

Capacity of the reservoir at the start = 5 Mcum

Capacity of the reservoir at the end

(*i.e.*, when 0.5 Mcum of sediment is filled up) = 4.5 Mcum

$\therefore$ Capacity/inflow at the start $= \dfrac{5\text{ Mcum}}{15\text{ Mcum}} = 0.333$

η at start = 95%. Capacity/inflow at the end $= \dfrac{4.5}{15} = 0.30$

η at the end of the interval = 95%. Average η = 95%

$\therefore$ Sediment load trapped/yr.

$= 0.0375 \times 95\% = 0.035625$

$\therefore$ No. of years during which 0.5 Mcum of sediment will get trapped

$= \dfrac{0.5}{0.035625}$ years = 14.04 years ; say **14 years.**

Hence, after 14 years, 10% reservoir capacity will get filled up. **Ans.**

18.14. Estimating Sediment Load likely to Enter a Proposed Reservoir

The quantum of sediment flowing into a dam reservoir along with river runoff primarily depends upon the erosion characteristics of its catchment and the characteristics of the rainfalls that produce the run off entering the reservoir.

The sediment is basically produced by rains by the process of sheet erosion. The flowing water is the most active agent for erosion of soil from the land. Other agents like *wind, gravity, ice* and *human activities* do help in the erosion process. The rain drops in itself, loosen the soil perticles and break the soil lumps. The action of flowing sheet of water on the land surface helps in eroding the top soil from the ground surface and transport it down to the channels. The erosion caused by rainfall and runoff, thus, constitutes of the following two parts :

(1) **Sheet erosion.** It includes the detachment of geological material from the land surface by the impact of raindrops, and its subsequent removal by overland flow; and

(2) **Channel erosion.** It includes river bank erosion and transportation of the materials by concentrated flow.

Human activities, like *overgrazing of grass land, cutting of forests, forest fires, ploughing of land,* and various *mining & other excavations* etc. have magnified the problem of water erosion in river channels.

18.14.1. Factors Affecting the Erodibility of a Soil. The factors which effect the erodibility of a soil are given below :

(*i*) *Particle size of soil.* Larger the size of soil particles, the lesser would be its chances for erosion.

(*ii*) *Land slope.* The greater is the land slope, the greater is the action of erosive agents, the optimum being at 40° slope.

(*iii*) *Vegetation.* The thicker is the vegetation cover over a soil, the lesser will be the scope of erosion of the soil from the area.

(*iv*) *Presence of Salt and Colloidal Matter in the Soil.* The binding materials like kaolinite, montmorillonite, biotite, etc., do help in increasing the force of cohesion between the soil particles, thereby reducing the erodibility of the soil.

(*v*) *Moisture Content of Soil.* The greater is the moisture content of the soil, the lesser is the scope of its erosion.

(*vi*) *Soil compaction.* The higher is the compaction of soil, the lesser is the chance of its erosion.

(*vii*) *Soil Properties.* The characteristics of the soil, such as soil texture, structure, stratification, permeability, composition etc. do affect the soil binding, which in turn will neutralise the force of weathering agents.

(*viii*) *Human Activities.* Human activities on the land like mining, agricultural operations, construction of projects, land use, etc. do increase the erosion from the given land.

(*ix*) *Rainfall characteristics.* The intensity, duration, quantity and distribution of rainfall over space and time are some of the important factors that affect sediment yield.

18.14.2. Estimation of Sheet Erosion. The sediment yield of a reservoir basically depends on sheet erosion which can be estimated by the following empirical equations :

(1) *Musgrave Equation.* Musgrave (1947) suggested the following equation to compute the annual gross sheet erosion from a catchment, on the basis of 19 widely scattered research stations in USA :

$$E = CR \cdot (S_0/10)^{1.35} (L/72)^{0.35} (P_{30}/1.25)^{1.75} \quad ...(18.15)$$

where E = Erosion of soil lost from the catchment in inches/year

C = the *soil erosion rate,* which varies from 0.43 to 0.53 inch/year depending on the soil type (which depends on texture and permeability of soil)

R = the *cover factor,* which varies from 0.95 for poorly covered land to 0.10 for row crops

S_0 = Land slope in percentage, the default being 10%

L = Length of the land slope in feet

P_{30} = The max. rainfall in inches having 30 min. duration and of 2 year frequency.

It was also stipulated that the erosion value of E computed above, in inch/year, can be multiplied by 150 to obtain the erosion value in ton/year/acre.

(2) *Universal Equation.* Agriculture Research Service of U. S. Deptt. of Agriculture developed a universal equation (1961) to predict erosion from **small catchments.** This universal equation is given as :

$$E_a = R_f \cdot K \cdot (LS)\, C_m \cdot P \quad ...(18.16)$$

where E_a = Average soil loss in ton/acre/year.

(*i*) R_f is **Rainfall-run off factor.** Its value takes into account the effect of rain drop impact as well as the resulting amount and rate of run off. It should also include the cumulative effects of many moderate sized storms as well as the effects of occasional severe storms.

The value of R_f is generally taken as equal to EI (**rainfall erosion index**), which is computed as :

$$E \cdot I = E \times I_{30} \quad ...(18.17)$$

where E = storm energy in 100 ft-ton/acre/inch

I_{30} = Max. 30-minute intensity in inch/hr.

The value of E is further computed as :

$$E = \frac{1}{100}\left[916 + 331 \log I\right] \quad ...(18.18)$$

where E = Storm energy in 100 ft-ton per acre per inch.

I = Storm intensity of the given storm with a limit of 3 inch/hr, since median drop size does not continue to grow beyond this limit.

The *EI* value can, thus, be computed by eqn (18.17) for a given storm. For a specified period, the individual storm *EI* values can be summed up, which provide a numerical measure of the erosive potential of the rainfall within that period (one year). *In this manner, the average annual total of the storm EI values in a particular area is obtained, which is called the rainfall-erosion index for that area,* which equals R_f.

(*ii*) K is **Soil erodibility factor.** Its value is determined experimentally for the given soil type. Representative values for different types of soils have been worked out and listed for different regions of USA by Soil Conservation Service of USA, as shown in Table 18.29.

Table 18.29. Computed K Values for Soils on Erosion Research Stations (After Wischmeier and Smith, 1978).

Soil	*Source of Data*	*Computed K*
Dunkirm silt loam	Geneva, NY	0.69[a]
Keene silt loam	Zanesville, OH	0.48
Shelby loam	Bethany, MO	0.41
Lodi loam	Blacksburg, VA	0.39
Fayette silt loam	LaCrosse, WI	0.38[a]
Cecil sandy clay loam	Watkinsville, GA	0.36
Marshall silt loam	Clarinda, IA	0.33
Ida silt loam	Castana, IA	0.33
Mansic clay loam	Hays, KS	0.32
Hagerstown silty clay loam	State College, PA	0.31[a]
Austin clay	Temple, TX	0.29
Mexico silt loam	McCredie, MO	0.28
Honeoye silt loam	Marcellus, NY	0.28[a]
Cecil sandy loam	Clemson, SC	0.28[a]
Ontario loam	Geneva, NY	0.27[a]
Cecil clay loam	Watkinsville, GA	0.26
Boswell fine sandy loam	Tyler, TX	0.25
Cecil sandy loam	Watkinsville, GA	0.23
Zaneis fine sandy loam	Guthrie, OK	0.22
Tifton loamy sand	Tifton, GA	0.10
Freehold loamy sand	Marlboro, NJ	0.08
Blath flaggy silt loam with surface stones > 2 inches removed	Arnot, NY	0.05[a]
Albia gravelly loam	Beemerville, NJ	0.03

a Evaluated from continuous fallow. All others were computed from rowcrop data.

For soils containing less than 70% of silt and very fine sand, K can be computed as :

$$K = \frac{1}{100}\left[2.1\, M^{1.14}\,(10^{-4})\,(12-a) + 3.25\,(b-2) + 2.5\,(c-3)\right] \quad ...(18.19)$$

where M = particle size parameter, defined as percent silt & very find sand (size 0.1 mm-0.002 mm) times the quantity (100-percent clay)

a = percent organic matter

b = soil texture code used in USDA soil classification

c = profile permeability class

[**Note :** When the silt fraction does not exceed 70%, erodibility varies approximately as the 1.14 power of M, but addition of organic matter content, soil structure, and profile-permeability class as done in Eqn. (18.19) improves the prediction accuracy.]

(*iii*) (LS) = **Topographic Factor.** It is the ratio of soil loss per unit area from a field slope to that from a 72.6 ft length of uniform 9% slope under otherwise identical conditions. For a specified slope and its length, LS can be computed as :

$$(LS) = \left(\frac{\lambda}{72.6}\right)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065) \qquad ...(18.20)$$

where λ = slope length in ft

θ = angle of slope

M = 0.5 if the per cent slope is 5 or more

m = 0.4 on slopes of 3.5 to 4.5%

m = 0.3 for slopes of 1 to 3%

m = 0.2 for uniform slopes of less than 1%.

(*iv*) C_m = **Soil Cover and Management Factor.** It measures the combined effect of all inter-related cover and management variables, including the type of vegetation, plant spacing, the stand, the quality of growth, crop sequence, tillage practices, crop residues, incorporated residues, land use residues, fertility treatment, etc.

Values of C_m for pasture, range, idle land, and woodland for a combination of cover conditions are given in Table 18.30.

Table 18.30. Factor C_m for permanent Pasture, Range, and Idle Land (After Wischmeier and Smith 1978).

Vegetative canopy		*Cover that contracts the soil surface*						
Type and height[b]	*Percent cover*[c]		*Percent ground cover*					
		Type[d]	*0*	*20*	*40*	*50*	*80*	*95+*
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.091	0.043	0.011
Tall weeds or short brush with the average drop fall height of 20 in.	25	G	0.36	0.17	0.09	0.038	0.013	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.120	0.003
		W	0.26	0.16	0.11	0.076	0.039	0.001
	75	G	0.17	0.10	0.06	0.032	0.011	0.003
		W	0.17	0.12	0.09	0.068	0.038	0.011
Appreciable brush or bushes, with the average drop fall height of 6 ½ ft	25	G	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.087	0.042	0.011
	50	G	0.34	0.16	0.08	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.082	0.042	0.011
	75	G	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.078	0.040	0.011
Trees, but no appreciable low brush Average drop fall height of 13 ft	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.089	0.042	0.011
	50	G	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.087	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.084	0.041	0.011

[a] The listed C values assume that the vegetation and mulch are randomly distributed over the entire area.

[b] Canopy height is measured as the average fall height of water drops falling from the canopy to the ground. Canopy effect is inversely proportional to the drop fall height and is negligible if fall height exceeds 33 ft.

[c] Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[d] G: cover at the surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 in. deep. W: cover at the surface is mostly broadleaf herbaceous plants (as weeds with little lateral-root network near the surface) or undecayed residues or both.

(v) P = **Support Practice Factor.** Its value depends upon crop land practice such as contour tillage, *strip cropping on the contour*, and *terrace systems*. Values of P have been given for each of these practices by Wischmeier and Smith (19 7 8). In general, tillage and planting on the contours reduce erosion. Table 18.31 gives P values for contouring.

Table 18.31. P values (Eq. 18.16) and Slope-Length Limits for contouring (Wischmeier & Smith)

Land Slope %	*P Value*	*Maximum Length* (feet)*
1 – 2	0.60	400
3 – 5	0.50	300
6 – 8	0.50	200
9 – 12	0.60	120
13 – 16	0.70	80
17 – 20	0.80	60
21 – 25	0.90	50

* Limit may be increased by 25% for residue cover after crop seedlings will regularly exceed 50%.

Since it is difficult to get proper data for the use of Eq. (18.16) for a developing country like India, this equation is not popular in developing countries.

18.14.3. Sediment Measurement by Sample Recorder. The sediment produced by sheet erosion from a catchment may not always reach at the point of measurement; *i.e.* the site of dam reservoir. Some part of sediment may be deposited en-route. *The ratio between the yield of sediment at the measuring site and the gross erosion in the catchment is called the* **sediment delivery ratio.**

Thus, the **sediment yield** is the gross sediment yield minus the quantity of sediment deposited en route. The sediment yield, infact, is important, since it is this sediment which will get deposited in the reservoir, affecting its useful life.

The *sediment yield of a reservoir* can be calculated either by using an appropriate empirical equation, out of the ones developed by various investigators; or by developing a rational appropriate relation between inflow and sediment, on the basis of actual measurements of sediment load at the site of the reservoir. The continuous measurements of suspended load and bed load at the reservoir site for a number of years will help in developing a rational relation (either a mathematical equation or a graphical curve) between sediment and the inflow into the reservoir. We will first of all discuss the method of the actual measurement of sediment and developing an appropriate relation between sediment yield (q_s) and the river discharge (Q) at the given site.

18.14.3.1. Measuring suspended sediment load. Sediment samplers are used to measure suspended sediment at a given site on the river to obtain the most reliable results of sediment yield. The bed load should also be calculated either by using the available empirical equations*, or on an adhoc basic of 2.5-25% of the suspended load, as to calculate the total sediment load (sum of suspended load and bed load).

A typical sediment sampler used in India is shown in Fig. 18.26. A depth integrating hand sampler used for small streams is also shown in Fig. 18.27.

* Please see article 4.9.

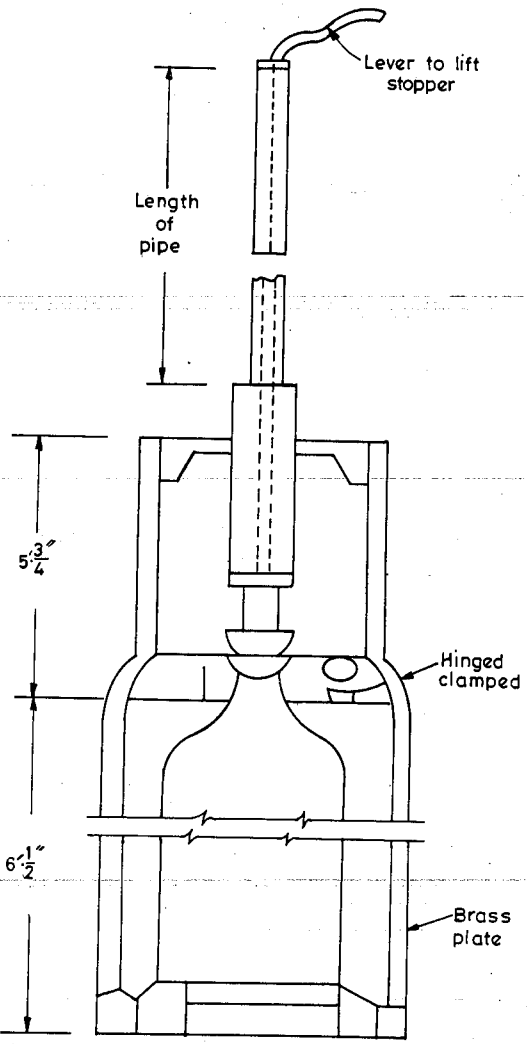


Fig. 18.26. Line diagram of a typical sediment sampler (Punjab bottle sampler).

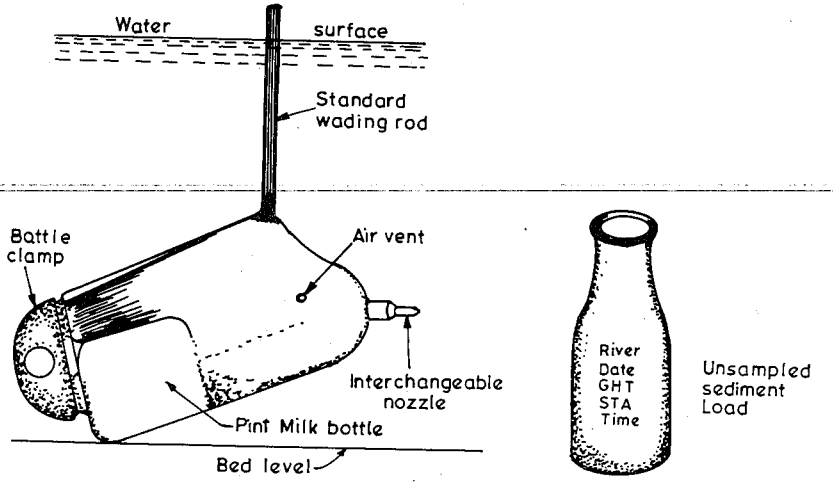


Fig. 18.27. AUS.DH-48 depth-integrating hand sampler for small streams.

The sampler is taken into the stream to a depth of 0.6y below the water surface, or at two depths at 0.2y and 0.8y depths to collect samples of sedimented water in the sampler bottle. The collected sample of sedimented water is then analysed in the laboratory either by a *gravimetric method*, or by *hydrometric method* to determine the quantum of coarse sediment (particle size higher than 0.2 mm), *medium size sediment* (particle size between 0.075 mm to 0.2 mm) and of *fine sediment* (particle size less than 0.075 mm), separately*. Their sum will indicate the total sediment load present in the given volume of water sample. Sediment load present in the water sample is then expressed in ppm (parts for million) as :

$$\text{Sediment load in ppm} = \frac{\text{Dry mass of sediment}}{\text{Total mass of original sample including the mass of sediment \& of water}} \times 10^6 \quad ...(18.21)$$

This value can be converted into t/day by multiplying the average unit wt. of sediment (say 1.2 t/m^3) with the total volume of daily inflow in m^3.

When a large number of such sample records become available for the given site, then a curve can be plotted between the sediment load (q_s) in t/day on x-axis and daily discharge in m^3/s on y-axis, as to obtain a curve known as *sediment rating curve,* as shown in Fig 18.28. Such a plotting is usually done on a log-log paper.

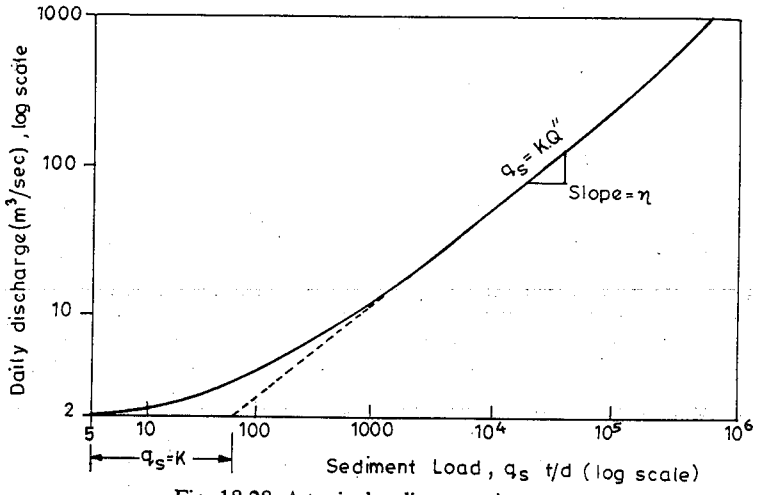


Fig. 18.28. A typical sediment rating curve.

When a standard sediment rating curve is established for a given site, the sediment yield (q_s) can be read out by simply knowing the discharge rate only. However, care must be taken to see that for different seasons of the year, different curves will have to be developed, since the sediment yield of a basin may vary with the season. The curve, for say March (non-monsoon period) may be entirely different from the curve for the month of say August (monsoon season).

* The sample collected by the sampler is first passed through a BSS-100 sieve and the coarse particles retained are taken out and oven dried. Thus, the quantity of coarse sediment (higher than 0.2 mm size) is obtained. Sedimented water passing 200 μ sieve is allowed to stand for 20 minutes, so that the finer particles settle down. The settled mass is removed by the process of decantation (pouring out water from the settled tank). The settled residue is dried and weighed, as to get the mass of fine sediment (0.075 mm – 0.2 mm). Poured water contains sediment of still finer particles. To isolate this, the sample is filtered through a filter paper, and the quantity retained therein is dried and weighed. This gives the mass of fine sediment (particle size < 0.075 mm). The summation of all the three masses will give the total mass of sediment.

The sediment rating curves can be used to compute daily sediment load for the given daily discharge values, and their summation can give us the monthly or the annual sediment load.

18.14.3.2. Mathematical equation for a sediment rating curve. The sediment rating curve (straight line portion) giving sediment load in tonnes/day (q_s) w.r. to daily discharge (Q) in m^3/sec can be expressed by a mathematical equation of the form :

$$q_s = K \cdot Q^n \qquad ...(18.22)$$

Taking log on both sides,

$$\log q_s = \log K + n \cdot \log Q$$

This equation is similar to the form

$$y = a + bx,$$

where $\log q_s$ is plotted on y-axis and $\log Q$ on x-axis.

$$n = b \qquad ...(18.23)$$

$$\log K = a \qquad ...(18.24)$$

A mathematical solution for such an equation can be obtained by the statistical **method of least squares,** where in the various known values of x (*i.e.* $\log Q$) and y (*i.e.* $\log q_s$) are analysed to estimate a and b values as :

$$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.25)$$

$$b = \frac{N \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2} \qquad ...(18.26)$$

By computing values of a and b, values of n and K become known to finally compute the relation between q_s and Q by Eq. (18.22).

The use of this method will become clear when we solve example 18.18.

Example 18.18. *A reservoir has the following sediment and discharge data :*

Year	*81*	*82*	*83*	*84*	*85*	*86*	*87*	*88*
Discharge ($M\,m^3$)	*1430*	*3850*	*2050*	*6510*	*2880*	*1120*	*6050*	*2220*
Sediment load (M.t) as measured by silt sampler	*2.65*	*5.82*	*3.60*	*7.15*	*5.22*	*1.95*	*6.88*	*3.94*

Calculate the average total sediment load/year/100 sq. km of the catchment at the site. Develop a regression relation and predict the total and observed sediment yield for the inflow of 3450 $M.m^3$ for the year 1978. Take the catchment area at the site as 3050 sq. km. What is the total sediment yield for 100 years ? Assume bed load as 10% of suspended load.

Solution. Average sediment load measured by silt sampler

$$= \frac{2.65 + 5.82 + 3.6 + 7.15 + 5.22 + 1.95 + 6.88 + 3.94}{8}$$

$$= \frac{37.21}{8} \text{ Mt/year} = 4.651 \text{ Mt/yr.}$$

Since sediment load is measured by a silt sampler, the given values of sediment are of suspended load.

Now, assume Bed load = 10% of suspended load

$\therefore$ Total load = Suspended load + Bed load

$\therefore$ Total sediment load = $1.1 \times 4.651 = 5.116$

or $(q_s) = 5.116$ Mt/yr.

Sediment deposited in a reservoir consolidates gradually due to the increasing silt load on it every year and the wt. of water above it. Assuming the average sediment unit weight (sediment in a reservoir consists of sand, silt and clay in water) as 1.2 t/m^3, we have

$\therefore$ Total silt load/year $= 5.116$ Mt

$$q_s = \frac{5.116}{1.2} \text{ M·m}^3 = \mathbf{4.263 \text{ M m}^3}$$

Catchment area $= 3050$ sq. km

$\therefore$ Sediment load/100 sq. km/year $= \dfrac{4.263 \text{ M m}^3}{3050} \times 100$

$= \mathbf{0.1397 \text{ M m}^3/100 \text{ sq. km/year}}$ **Ans.**

Total sediment yield in 100 years = 4.263 M m^3/yr. × 100 yr. = **426.3 M m^3** **Ans.**

In order to develop a regression type non-linear equation between total sediment load (q_s) and discharge (Q), represented as $q_s = K \cdot Q^n$, we carry out the required calculations in table 18.32 to evaluate the values of a and b by Eqn. (18.25) and (18.26), as :

Table 18.32. Computations to develop Equation $y = a + bx$ by the method of least squares.

year	*Discharge* Q ($M \cdot m^3$)	*Suspended Sediment measured in M.t*	*Total sediment i.e.* q_s *in M m*^{3} $col\,(3) \times \frac{1.1^*}{1.2}$	$\log Q = x$ $\log col\,(2)$	x^2 $(col\,5)^2$	$\log q_s = y$ $\log col\,(4)$	$x \cdot y$ $col\,(5) \times col\,(7)$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
81	1436	2.65	2.429	3.1553	9.9561	0.3855	1.2164
82	3850	5.82	5.335	3.5855	12.8555	0.7271	2.6070
83	2050	3.60	3.300	3.3118	10.9677	0.5185	1.7172
84	6510	7.15	6.554	3.8136	14.5433	0.8165	3.1138
85	2880	5.22	4.785	3.4594	11.9674	0.6799	2.3520
86	1120	1.95	1.788	3.0492	9.2977	0.2522	0.7690
87	6050	6.88	6.307	3.7818	14.3017	0.7998	3.0247
88	2220	3.94	3.612	3.3464	11.1981	0.5577	1.8663
			Σ	27.503	95.0875	4.7372	16.6664

* q_s (Total sediment in Mm^3) $= \dfrac{q_s \text{ in M} \cdot \text{t}}{1.2 \text{ t/m}^3} \times (1.1)$** $= \left(col\,(3) \times \dfrac{1.1}{1.2} \right)$

** Total sediment = 1.1 (Observed sediment by soil sampler)

Using $$a = \frac{\Sigma y \cdot \Sigma x^2 - \Sigma x \cdot \Sigma xy}{N\Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.25)

we get $$a = \frac{4.7372 \times 95.0875 - 27.503 \times 16.6664}{8 \times 95.0875 - (27.503)^2}$$

or $$a = \frac{-7.9275}{4.2850} = -1.8500$$

But $\log K = a = -1.8500 \quad \therefore \quad K = 0.014$

Also $$b = \frac{N \cdot \Sigma xy - \Sigma x \cdot \Sigma y}{N \cdot \Sigma x^2 - (\Sigma x)^2}$$ ...(*i.e.* 18.26)

$$= \frac{8 \times 16.6664 - 27.503 \times 4.7372}{8 \times 95.0875 - (27.503)^2} = \frac{3.0440}{4.2850} = 0.7104$$

But $b = n = 0.7104$

Regression equation is hence given as :

$$q_s = K \cdot Q^n = 0.014\,(Q)^{0.7104}$$ **Ans.**

Total sediment yield for the year 1978 having inflow of 3450 M m^3

$$= 0.014\,(3450)^{0.7104} = \mathbf{4.565\ M\ m^3}$$

$$= 4.565\ \text{M m}^3 \times 1.2\ \text{t/m}^3 = \mathbf{5.48\ Mt}$$ **Ans.**

Observed or suspended sediment load for the year 1978

$$= \frac{5.48}{1.1}\ \text{Mt} = \mathbf{4.98\ Mt}$$ **Ans.**

18.14.3.3. Empirical equations for total sediment yield. The following empirical equations have been developed by several investigators, for estimating the annual sediment yield of a reservoir.

(1) *Swami's Regression Equation.* Swamy and Garde (1977) have proposed a relation correlating the cumulative volume of sediment deposited in a reservoir with the cumulative volume of water inflow, and initial bed slope of the river, as

$$V_s = C \cdot B \cdot (V_{ci})^{0.94}\,(S_0)^{0.84}$$ ...(18.27)

where V_s = cumulative vol. of sediment deposited in the reservoir in M · m^3

C = Regression constant with safe value of the order of 1.16. However a value less than 1.16 may be adopted depending on the reservoir

B = Width of the reservoir at full reservoir level in m

V_{ci} = Cumulative vol. of inflow per unit width B of the reservoir

S_o = Bed slope of the river.

(2) *Jogelkar's Equation.* An equation proposed by Jogelkar (1960) is given as :

$$Q_s = 0.59\,(A)^{-0.24}$$ ...(18.28)

where Q_s = Annual silting rate in M · m^3 per 100 sq. km of catchment area

A = Catchment area in sq km

(3) *Khosla's Equation.* Khosla (1953) has proposed the following empirical equation.

$$Q_s = 0.323\,(A)^{-0.28} \qquad ...(18.29)$$

where Q_s = Annual siltation rate in $M \cdot m^3/100$ s.q km/year

A = Catchment area in sq. km.

This equation always gives lower estimate of sediment yield at a site.

(4) *Varshneys Equations.* Varshney and Raichur have proposed the following equations for calculating sediment yield for an ungauged basin.

(*i*) *Up to 130 sq. km catchment for mountainous rivers*

$$Q_s = 0.395\,(A)^{-0.311} \qquad ...(18.30)$$

where Q_s = Annual sediment yield rate in $M \cdot m^3$ per 100 sq. km of catchment

A = catchment area in km^2.

(*ii*) *Rivers draining plain area up to 130 sq. km*

$$Q_s = 0.392\,(A)^{-0.302} \qquad ...(18.31)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*iii*) *For area greater than 130 sq. km for North Indian catchment*

$$Q_s = 1.534\,(A)^{-0.264} \qquad ...(18.32)$$

Q_s & A have the same meaning as in Eq. (18.30).

(*iv*) *For South Indian Rivers up to 130 sq. km*

$$Q_s = 0.46\,(A)^{-0.468} \qquad ...(18.33)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(*v*) *For areas greater than 130 sq. km for South Indian catchments*

$$Q_s = 0.277\,(A)^{-0.194} \qquad ...(18.34)$$

Q_s & A have the same meaning as in Eqn. (18.30).

(5) *Using Known Data of Similar Catchment.* Sediment yield of an unmeasured watershed Q_{s_2} can be computed from sediment yield of measured catchment Q_{s_1} of similar topography, land cover and land use, on area proportion basis, as

$$Qs_2 = Qs_1 \left(\frac{A_2}{A_1}\right) \qquad ...(18.35)$$

Example 18.19. *Estimate the sediment load in tonne at the proposed dam site in North India with the following data using various empirical equations :*

Catchment area = 1830 sq. km

Width of reservoir at FRL = 560.0 m

River slope at the dam site = 0.006

Average inflows at the site are as follows :

Year	*1982*	*1983*	*1984*	*1985*	*1986*	*1987*	*1988*	*1989*	*1990*
Inflow $M \cdot m^3$	*2210*	*1290*	*1640*	*1780*	*2150*	*1980*	*2540*	*1285*	*1620*

Assume annual siltation rate per 100 sq. km from a similar catchment of 3050 sq. km to be 10.35 M · m³/100 sq. km.

Solution. The sedimentation rate is worked out by using various equations as described in the previous article.

(1) *By Swamy's Regression Method :* The computation is carried out in Table 18.28 by using Eqn. (18.27) as :

$$V_s = C \cdot B\,(V_{ci})^{0.94}\,(S_0)^{0.84}$$

where $C = 1.16$

$B = 560$ m

$S_0 = 0.006$

$\therefore$ V_s = cumulative vol. of sediment deposited in M · m³

$$= 1.16 \times 560\,(V_{ci})^{0.94} \times (0.006)^{0.84}$$

$$= 8.837\,(V_{ci})^{0.94}$$

where V_{ci} is the cumulative vol. of annual inflows per unit width B of reservoir, over the given years.

The computations are carried out in Table 18.33, which is self explanatory.

Table 18.33. Computation of sediment Load by Swamy's Regression Method

S. No. (1)	*Year* (2)	*Inflow in M · m³* (3)	*Cumulative inflow in M m³* (4)	V_{ci} = *cumulative inflow per unit width B of reservoir* = $\frac{col\,(4)}{560}$ (5)	$V_s = 8.837\,(V_{ci})^{0.94}$ $= 8.837\,(col\,5)^{0.94}$ M · m³ (6)
1	1982	2210	2210	3.946	32.11
2	1983	1290	3500	6.250	49.48
3	1984	1640	5140	9.179	71.01
4	1985	1780	6920	12.357	93.91
5	1986	2150	9070	16.196	121.10
6	1987	1980	11050	19.732	145.80
7	1988	2540	13590	24.268	177.11
8	1989	1285	14875	26.563	192.81
9	1990	1620	16495	29.455	**212.48**

Q_s = sedimentation rate per 100 sq. km

$$= \frac{V_s \text{ (i.e. cumulative vol. of silt)}}{\text{No. of years (i.e. 9)}} \times \frac{100 \text{ sq. km}}{C \cdot A \text{ of } 1830 \text{ km}^2}$$

$$= \frac{212.48}{9} \times \frac{100}{1830}$$

or $Q_s = \mathbf{1.29\ M\ m^3/100\ sq.\ km}$ **Ans.**

(2) *By Jogelkar's Equation*

$$Q_s = 0.59\ (A)^{-0.24}$$

$$= 0.59\ (1830)^{-0.24}$$

$$= \mathbf{0.10\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(3) *By Khosla's Equation*

$$Q_s = 0.323\ (A)^{-0.28}$$

$$= 0.323\ (1830)^{-0.28}$$

$$= \mathbf{0.04\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(4) *By Varshneys Equation for Norths Indian catchment exceeding 130 sq. km.*

$$Q_s = 1.534\ (A)^{-0.264}$$

$$= 1.534\ (1830)^{-0.264}$$

$$= \mathbf{0.211\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

(5) *From similar catchment.*

$Q_{s_1} = 4.56$ M m^3/100 sq. km

$A_1 = 3050$ sq. km

$A_2 = 1830$ sq. km

$$Q_{s_2} = Q_{s_1} \times \left(\frac{A_2}{A_1}\right)$$

$$= 0.35 \times \left(\frac{1830}{3050}\right) \text{M m}^3/100 \text{ sq. km}$$

$$= \mathbf{0.21\ M\ m^3/100\ sq.\ km} \quad \textbf{Ans.}$$

18.15. Reservoir Sedimentation Studies on Existing Reservoirs

Sedimentation of storage reservoirs is a natural process, since large part of the silt eroded from the catchment and transported by the river, gets deposited on the bed of the reservoir. This causes reduction in the live as well as dead storage capacities of the reservoir. Progressive loss of capacity due to sediment accumulation results in reduced benefits and may even cause operational problems. *It, therefore, becomes necessary to monitor the sedimentation rates in the existing reservoirs at regular intervals, to help in planning and executing suitable remedial measures for controlling sedimentation in order to prolong the life of the reservoir and its benefits.*

Regular monitoring and updating of elevation-capacity curve of the reservoir immensely helps in better water management. With this aim, *conventional hydrographic surveys* are conducted at regular intervals at the existing reservoirs to determine the available capacities at different elevations, and to help compute the sedimentation volume at such regular intervals. Such conventional surveys will require computation

of water spread area at different water levels, and is quite a tedious and a costly process. **Remote sensing techniques** do offer a modern answer to the costly conventional surveys, as it offers a great potential for application in capacity evaluation of medium to large reservoirs. From the data provided by the remote sensing satellites, it has now been possible to compute loss of reservoir capacity due to sedimentation, and its distribution. The results obtained from this technique have been found to be quite comparable with those obtained from the costly and cumbersome conventional methods. One of the greatest advantage of this technique is that the capacity evaluation could be easily computed on yearly basis.

The methodology involved in this technique requires the use and analysis of the *satellite imageries* provided by the Remote Sensing satellites, which collect the data of the Earth surface features in different bands at regular intervals. In the case of Indian Remote Sensing Satellites IRS-1A and IRS-1B* (both identical satellites), this periodicity is 22 days for Indian sub-continent. These two satellites together are thus capable to provide us data of all our reservoirs at 11 days interval. The third satellite of this series IRS-1C has also been launched recently on 28.01.1996 and helps in providing better pictures even of cloud bound areas. IRS-1D has further been launched on 29.09.1997, through our first Indian rocket launcher.

Due to the water withdrawals from an existing reservoir, its water spread area goes on changing throughout the year. The reservoirs are generally full just after the monsoon period in October, and get depleted to almost dead storage/minimum drawdown level just before start of monsoon season in May or June, every year. The satellite data of various dates during the period from October to May, provide us an array of water spread areas between maximum water level (i.e. around the FRL) and the minimum reservoir level (i.e. around the dead storage level/minimum drawdown level). From the whole set of the satellite data, a few of them which are cloud free and of good quality and representative of the whole range of reservoir levels at close intervals, are selected for analysis.

The method of analysis depends upon the data products. The selected CCT's/FCC's of various dates are analysed for determining the water spread areas. The corresponding water levels are obtained from the daily gauge record of the reservoir. From these, the water volumes between two consecutive water levels are computed using Prismoidal or any appropriate formula. Volume of water below the minimum water level (as recorded by the satellite) and the "new zero"** elevation, are estimated based upon the previous hydrographic surveys. In case, these informations are not available in the hydrographic survey data, then the elevation-area relationship obtained from the hydrographic surveys as well as the one obtained from the satellite data interpretation, should be extended to get the new zero elevation, and then the volume between the minimum mapped water level and this new zero level is estimated. After this, the cumulative water volume at each reservoir level is computed, and then the revised elevation-area-capacity curve is drawn. By comparing the original area-capacity curve or any other such curve, the total sediment volume and its distribution can be computed.

18.16. Observed Sedimentation Rates for Various Important Indian Reservoirs

The observed sedimentation rates for various important dam reservoirs in India are indicated in col. (6) & (7) of Table 18.34.

* IRS-1A was launched on 19.03.1988, and IRS-1B was launched on 23.08.1991.

** The minimum reservoir bed level is raised due to sedimentation, which is termed as the new zero.

Table 18.34. Annual Sedimentation Rates of Various Indian Reservoirs

S. No. (1)	*Name and Location of reservoir* (2)	*Catchment area in 100 sq. km* (3)	*Capacity of reservoir at FRL in* $M \cdot m^3$ (4)	*Surface area at MRL* $M \cdot m^2$ (5)	*Annual sediment rate in ham/100 sq. m* (6)	*Annual volume of sediment deposit* $M \cdot m^3$ $\frac{col\ (6)}{100} \times col\ (3)$ (7)	*Dead storage capacity provided in* $M \cdot m^3$ (8)
1	Bhakra *i.e.* Govind Sagar) (HP)	568	9351	169	6.00	**34.08**	95
2	Gandhi Sagar (MP)	226	7413	660	10.08	22.71	586
3	Hirakund (Orissa)	834	8146	725	3.89	32.44	2318
4	Lower Bhawani (TN)	61.5	929	—	4.10	2.52	0
5	Maithan (DVC)	63	1275	—	13.02	8.20	165
6	Matatila (UP)	207.5	883	—	3.50	7.26	50
7	Mayurkshi (WB)	18.6	617	—	**20.09**	3.74	68
8	Nizam Sagar (AP)	216.94	715	130	6.57	14.25	—
8	Panchat (DVC)	111	1475	153	9.92	11.01	170
10	Ramganga (UP)	30.76	2448	—	17.30	5.32	395
11	Sriram Sagar or Shivaji Sagar (AP)	8.19	3454	—	15.20	1.24	849
12	Tawa (MP)	—	2312	—	8.10	—	263

18.17. Reservoir Losses

Huge quantity of water is generally lost from an impounding reservoir due to *evaporation, absorption,* and *percolation.* Depending upon which, the following losses may occur from such a reservoir :

1. *Evaporation losses ;*
2. *Absorption losses ;* and
3. *Percolation losses* or *Reservoir leakage.*

These losses are discussed below.

18.17.1. Evaporation Losses. The evaporation losses from a reservoir depend upon several factors, such as : *water surface area, water depth, humidity, wind velocity, temperature, atmospheric pressure* and *quality of water* as discussed in article 7.34.2.3. The evaporation loss from a reservoir under the given atmospheric conditions can be easily estimated by measuring the standard pan evaporation and multiplying the same by the pan coefficient. The pan coefficients and various types of pans in use are given in article 7.34.3.4.

The evaporation losses become very significant in a hot and humid country like India ; and realistic estimation of these losses is quite important. These losses in fact vary from place to place and from season to season, and hence monthly values of these losses are usually determined. Typical average values of these losses for North and South India are given in table 18.35.

Table 18.35. Monthly Reservoir Evaporation Losses

Month	*Losses in cm*	
	North India	*South and Central India*
January	7	10
February	9	10
March	13	18
April	16	23
May	27	25
June	24	18
July	18	15
August	14	15
September	14	15
October	13	13
November	9	10
December	8	10
Total	172	182

On the basis of a review conducted on 130 sample reservoirs, the Central Water Commission, in 1990, has, however, estimated the *average annual evaporation loss to be 225 cm ;* and the total water lost from all the existing reservoirs to be 27000 Mm^3 per annum. What a tremendous waste of precious water!

In order to control such large scale wastage of water, several methods have been devised by engineers and scientists. All these methods are based upon the efforts made to reduce the evaporation rate from the surface of the water bodies by physical or chemical means, since the basic meteorological factors affecting evaporation cannot be controlled under normal conditions. *The following methods are generally used for evaporation control :*

1. Wind breakers
2. Covering of the water surface
3. Reduction of the exposed water surface
4. Use of underground storage rather than the use of surface storage
5. Integrated operation of reservoirs
6. Use of chemicals for retarding the evaporation rate from the reservoir surface.

Out of all these methods, the last method has evoked the maximum response from all over the world, and has been considered to be the only practical solution for conservation of fresh water, inspite of its various limitations and disadvantages in high cost of application in normal conditions. The use of chemicals, called **Water Evapo-Retardants (WERs)**, for controlling the evaporation rate from the surface of reservoirs is therefore, discussed here in details.

A non toxic chemical, capable of forming a thin *monomolecular film* over the water surface, is generally spread over the reservoir water surface in powder, liquid or emulsion form. *The resulting film prevents energy inputs from the atmosphere, thus reducing evaporation. Such a film, however, allows the passage of enough air through it, to avoid any harmful effects on the aquatic life due to shortage of oxygen.*

Fatty alcohols of different grades like : Cetyl alcohol ($C_{16}.H_{33}.OH$) popularly called *hexa decanol*, Stearyl alcohol ($C_{18}.H_{37}.OH$) popularly called *Octadecanol*, and Behenyl alcohol ($C_{22}.H_{45}.OH$) called *docosanol*, or a mixture of these chemicals, have been generally used and found to be quite suitable. These chemicals should, however, be 99% pure for getting the desired properties of *monolayer*. National Chemical Laboratory, Pune, has developed one more compound by synthesising alkoxy ethanols.

In general, all such chemical compounds should possess the *following properties* :

(*i*) the chemical compound (WER) should be tasteless, odourless, non-toxic, non-inflammable, and should not produce any effect on the quality of water.

(*ii*) the chemical should easily spread and form an even compact cohesive and efficient monomolecular film on the water surface.

(*iii*) the thin film formed by the chemical should be pervious to oxygen and carbon dioxide, but tight enough to prevent escape of water molecules.

(*iv*) the thin film formed by the chemical should be durable, and should be able to re-seal itself, when broken due to external disturbances such as wind, waves, etc.

(*v*) the chemical and the film formed by it should not be adversely affected by the water borne bacteria, proteins and other impurities present in the water body.

The use of chemical WERs has, however not been found to be cost effective for mass scale use, and has further not been found to be suitable under the following conditions :

(*a*) when the wind velocities exceed 10 km/hr or so.

(*b*) when the temperature rises above 40°C or so.

(*c*) when the size of the water body is relatively large.

Development of cheaper WERs capable of withstanding higher wind speeds upto about 20 km/hr and having strong cohesive forces and properties of self spreading and re-uniting to maintain the monolayer in resilient state even at high wind velocities, is therefore of vital importance. Moreover, the life of the film formed, must be longer, so as to reduce the frequency of application to about 3 to 7 days from its present frequency of 24 hours. Development of such chemical WERs is the subject matter of present research.

Other long term evaporation control measures like *plantation of trees to act as wind breakers**, *reduction of exposed water surface by covers, underground storage of water, integrated operation of reservoirs,* etc. have been employed in some parts of the country. The effectiveness and economics of these methods are, however, yet to be established.

In India, the water conservation methods are presently being adopted only in draught prone and scarcity areas, since large scale use of such methods on all the reservoirs of the country is not found to be economical or practically unfeasible due to their large size and adverse meteorological factors.

18.17.2. Absorption Losses. These losses do not play any significant role in planning, since their amount, though sometimes large in the beginning, falls considerably as the pores get saturated. They certainly depend upon the type of soil forming the reservoir.

18.17.3. Percolation Losses or Reservoir Leakage. For most of the reservoirs, the banks are permeable but the permeability is so low that the leakage is of no

* Trees having lesser evapotranspiration should only be chosen and identified.

importance. But in certain particular cases, when the walls of the reservoir are made of badly fractured rocks or having continuous seams of porous strata, serious leakage may occur. Sometimes, pressure grouting may have to be used to seal the fractured rocks. The cost of grouting has to be accounted in the economic studies of the project, if the leakage is large.

18.18. Reservoir Clearance

The removal of trees, bushes and other vegetation from the reservoir area is known as reservoir clearance. It is an expensive operation and difficult to be justified on cost-benefit considerations. Non-clearance of such vegetation may lead to the following troubles :

(*i*) decay of organic material may create undesirable odours and tastes, and hence, becomes important for water supply reservoirs.

(*ii*) trees projecting above the water surface may create undesirable appearance, when the reservoir is to be used for recreation and tourism purposes.

(*iii*) bushes, trees, etc. will float and may create debris problems at the dam.

18.19. Selection of a Suitable Site for a Reservoir

It is almost impossible to select a perfect ideal reservoir site. But its selection is guided by the following factors :

(*i*) A suitable dam site is available. The cost of the dam is generally a controlling factor in the selection of a reservoir site.

(*ii*) The geological formations for the reservoir banks, walls, etc. should be such as to entail minimum leakage.

(*iii*) The geology of the catchment area should be such as to entail minimum water losses through absorption and percolation.

(*iv*) The site should be such that a deep reservoir is formed. A deep reservoir is preferred to a shallow one, because of lower land cost per unit of capacity, less evaporation loss, and less possibility of weed growth.

(*v*) The reservoir site must have adequate capacity.

(*vi*) Too much silt laiden tributaries should be avoided as far as possible.

(*vii*) The reservoir basin should have a deep narrow opening in the valley, so that the length of the dam is minimum.

18.20. Reservoir Induced Seismicity

During the recent times, a very strange phenomenon has been observed in several dams of the world. What actually has happened is that the reservoir basins or areas which were seismologically inactive, started showing seismic activities, as the reservoir was filled up with water. The magnitude of these earthquakes is found increasing with the filling of the reservoir, giving severe shocks when the reservoir is full. In all such cases, the epicentre is always seen to be located within the reservoir or along its border.

An important Indian example of such an earthquake has been in the case of Koyna dam (Maharashtra), which is situated in the seismologically inactive zone in Peninsular India. However, when the dam got ready and water started collecting (1962) in the reservoir, the earthquakes also started occurring (1963). The frequency and intensity of these earthquakes had gone on increasing, as more and more water went on collecting in the reservoir. On 10th December, 1967, the severest shock occurred with a magnitude of about 6.5, followed by three more shocks of decreasing magnitudes. The epicentres of all these shocks were traced to be within the reservoir area.

Some other examples of such earthquake sites, noticed outside India, are at *Lake Mead* (U.S.A.) ; *Grand Val Lake* (France) ; *Vorgorno Lake* (Switzerland) ; etc.

This phenomenon has been studied by various seismologists, and it has been suggested by some that in most such cases, *there must have existed some inactive faults in the reservoir basin area, which became active again due to the extra-ordinary load of the reservoir water,* thus causing displacements along these faults and consequently resulting in earthquakes. Some other seismologists have suggested that these earthquakes are caused due to *increased pore pressure* in the adjoining rocks, which lowers their shearing strength, resulting in the release of tectonic strain.

Based on these explanations, the various preventive methods suggested for preventing or reducing such earthquakes, include :

(*i*) *filling of the reservoir to a limited safe level ;*

(*ii*) *reducing pore pressure by draining out water from weaker adjoining rocks ; and*

(*iii*) *to actively explore the dam site for the absence of inactive faults before selecting the same.*

Such earthquakes, however, show a decreasing tendency with time. This phenomenon is indeed very complex and interesting, and still needs further research.

18.21. Economic Height of a Dam

The economic height of a dam is that height of the dam, corresponding to which, the cost of the dam per unit of storage is minimum. For this purpose, the estimates are prepared for construction costs, for several heights of the dam, somewhat above and below the level at which the elevation-storage curve shows a fairly high rate of increase of storage per unit rise of elevation, keeping the length of the dam moderate. The construction cost is found to increase with the dam height, as shown in Fig. 18.29.

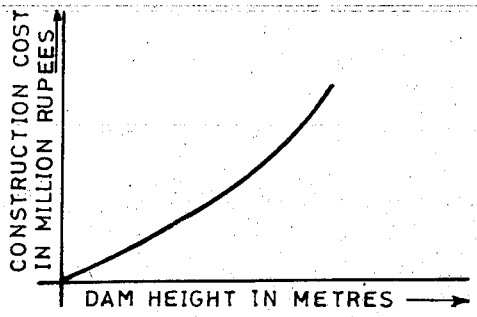


Fig. 18.29

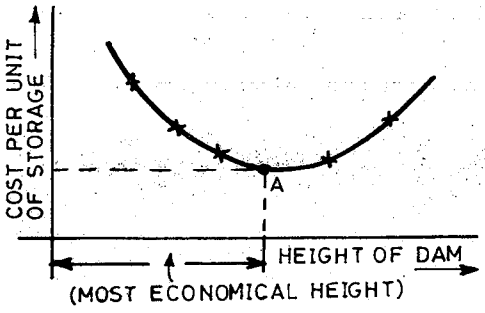


Fig. 18.30

For each dam height, the reservoir storage is known from the reservoir-capacity curve. The construction cost per unit of storage for all the possible dam heights can then be worked out and plotted, as shown in Fig. 18.30.

The lowest point A on this curve, gives the dam height for which the cost per unit of storage is minimum, and hence, most economical.

Example 18.20. *The construction costs for certain possible heights of a dam at a given site have been estimated and are tabulated in the table below. The storage capacity for all these dam heights are also given.*

S.No.	Height of the dam in metres	Construction cost in million Rs.	Storage in million cubic metres
(1)	(2)	(3)	(4)
1	10	4	50
2	20	8	110
3	30	12	180
4	40	18	250
5	50	27	350
6	60	39	500
7	65	50	600

Determine the most economical height of the dam from purely construction point of view.

Solution. The given table is extended, so as to workout the cost per million cubic metre of storage, as shown in col. (5) of Table 18.36.

Table 18.36

S.No.	Height of the dam in metres	Construction cost in M.Rs.	Storage in $M.m^3$	Cost per unit of storage $= \frac{Col.\ 3}{Col.\ 4}$
(1)	(2)	(3)	(4)	(5)
1	10	4	50	0.080
2	20	8	110	0.073
3	30	12	180	0.067
4	40	18	250	0.072
5	50	27	350	0.077
6	60	39	500	0.078
7	65	50	600	0.083

The cost per unit of storage is plotted against the height of the dam, as shown in Fig. 18.31. The most economical height is the lowest point of this curve, and it works out to be **30 metres. Ans.**

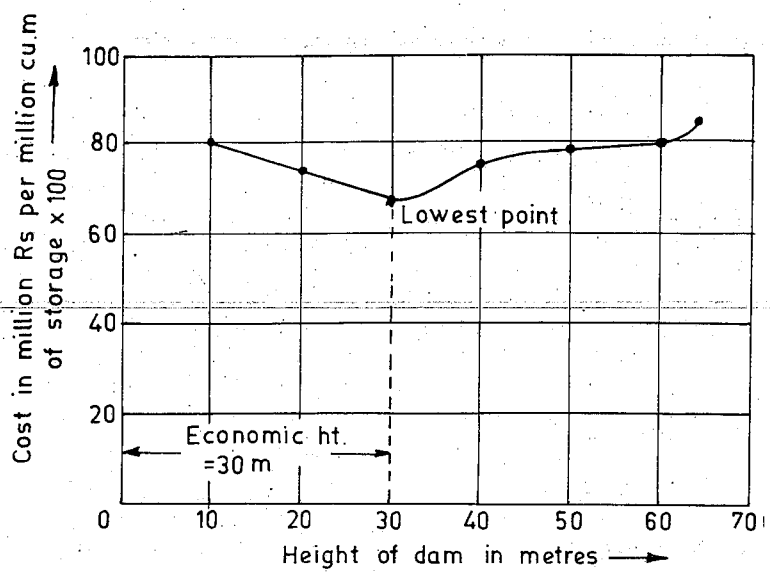


Fig. 18.31.

PROBLEMS

1. (*a*) What is meant by a 'Reservoir'? Discuss briefly the different types of reservoirs and the purpose served by each type.

(*b*) Describe briefly the techniques that are employed for computing the storage capacity of a reservoir for different water surface elevations.

2. (*a*) What is meant by a 'Flood control reservoir' ; and what are their different types ?

(*b*) Discuss with a neat sketch, the various storage zones of the dam reservoir.

(*c*) What factors you will keep in mind while selecting a suitable site for a dam reservoir?

3. (*a*) Differentiate clearly between the following :

(*i*) A flood control reservoir and a multipurpose reservoir.

(*ii*) A retarding basin and a storage reservoir.

(*iii*) Firm yield, design yield, and secondary yield of a reservoir.

(*b*) Briefly describe as to how you would fix the storage capacity of a reservoir and the height of the dam required for this storage. (*Madras University, 1976*)

4. (*a*) What is the relation between 'reservoir capacity' and 'reservoir yield'?

(*b*) How would you fix the capacity of a dam reservoir at a particular river site, provided the inflow pattern and demand pattern are known. Explain the mass curve method which is used for this purpose.

5. (*a*) Explain how the storage capacity of a reservoir is fixed.

(*Madras University, 1973, 1974*)

(*b*) Explain the mass curve method that can be used for determining :

(*i*) Reservoir capacity for fulfilling given demand.

(*ii*) Demand rate from a reservoir of a given capacity.

6. Discuss briefly and with necessary neat sketches, the demand patterns for the following types of reservoirs :

(*i*) Single purpose conservation reservoir.

(*ii*) Single purpose flood control reservoir.

(*iii*) Multipurpose reservoir.

7. (*a*) What is meant by 'flood routing through reservoirs'?

(*b*) Describe step by step procedure that you will adopt for flood routing computations required for reservoirs under 'trial and error method'.

8. Describe a method for routing flood water through a deep reservoir using the fundamental relation between inflow, outflow (discharge) and storage. Take $Q = \text{C.L.}\, H^{3/2}$ for the spillway.

(*U.P.S.C.,Engg. Services, 1974*)

9. The initial inflow in a reservoir and outflow over the spillway was 30 m^3/sec. During a storm, the following inflow rates were noted at the ends of successive half day periods : 240, 300, 240, 90, 30 and 30 m^3/sec. The relationship between discharge over the spillway (Q), and storage rate in the reservoir $\left(\frac{S}{T}\right)$ may be expressed by the equation

$$Q = \frac{1}{2} \cdot \frac{S}{T}.$$

Assuming that the average inflow rate during each half day period is equal to the average of the rates occurring at the beginning and at the end of the period, find the outflow at the end of half hour period.

Hence plot to a suitable scale the inflow and outflow hydrographs.

10. What are the factors on which the rate of silting of an impounding reservoir depends? What is trap efficiency?

Discuss the principal measures that should be undertaken to control the inflow sediment to an impounding reservoir.

An impounding reservoir had original storage capacity for 738 ha-m. The drainage area of the reservoir is 80 sq. km, from which, annual sediment discharges into the reservoir at the rate 0.1153 ha-m

per sq. km. of the drainage area. Assuming the trap efficiency as 80 per cent, find the annual capacity loss of the reservoir in per cent per year. *(U.P.S.C., Engg. Services, 1969)*

[**Ans.** 10%]

11. Write short notes on any four of the following :

(*i*) Reservoir losses (*ii*) Reservoir clearance

(*iii*) Economic height of a dam

(*iv*) Cost benefit considerations in planning dam reservoirs.

[**Note.** Please See Chapter 20 of "Hydrology and Water Resources Engineering"]

(*v*) Reservoir sedimentation and its control

(*vi*) Density currents

(*vii*) Trap efficiency

(*viii*) Estimating the life of a reservoir.

19

Design and Construction of Gravity Dams

19.1. Definition, etc.

A gravity dam has been defined as a structure which is designed in such a way that its own weight resists the external forces. This type of a structure is most durable and solid, and requires very little maintenance. Such a dam may be constructed of masonry or concrete. However, concrete gravity dams are preferred these days and mostly constructed. They can be constructed with ease on any dam site, where there exists a natural foundation strong enough to bear the enormous weight of the dam. Such a dam is generally straight in plan, although sometimes, it may be slightly curve. The line of the upstream face of the dam, or the line of the crown of the dam if the upstream face in sloping, is taken as the reference line for layout purposes, etc. and is known as the *Base line of the dam* or the '*Axis of the Dam*'. When suitable conditions are available, such dams car be constructed up to great heights. The highest gravity dam in the world is Grand Dixence Dam in Switzerland (284 m), followed by Bhakra dam in India (226 m); both are of concrete gravity type. The ratio of base width to height of all these structures is less than 1 : 1.

19.2. Typical Cross-section

A typical cross-section of a concrete gravity dam is shown in Fig. 19.1. The upstream face may by kept throughout vertical or partly slanting for some of its length, as shown. A drainage gallery is provided in order to relieve the uplift pressure exerted by the seeping water.

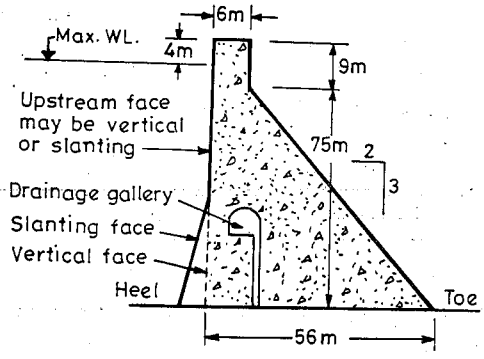


Fig. 19.1. A typical cross-section of a concrete gravity dam.

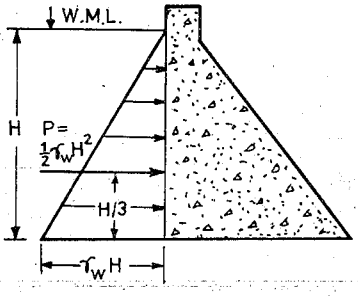
19.3. Forces Acting on Gravity Dam

The various external forces acting on a gravity dam may be :

(1) Water Pressure
(2) Uplift Pressure
(3) Pressure due to earthquake forces
(4) Silt Pressure
(5) Wave Pressure
(6) Ice Pressure
(7) The stabilising force is the weight of the dam itself.

An estimation and description of these forces is given below :

(1) **Water Pressure.** Water pressure (P) is the most major external force acting on such a dam. The horizontal water pressure, exerted by the weight of the water stored on the upstream side on the dam can be estimated from rule of hydrostatic pressure distribution ; which is triangular in shape, as shown in Fig. 19.2 (a) and (b). When the upstream face is vertical, the intensity is zero at the water surface and equal to $\gamma_w H$ at the base ; where γ_w is the unit weight of water and H is the depth of water : as shown in Fig. 19.2 (a). The resultant force due to this external water $= \frac{1}{2} \gamma_w H^2$, acting at $H/3$ from base.



Where γ_w = unit weight of water
9.81 kN/m^3 = 1000 kgf/m^3
Fig. 19.2. (a)

When the upstream face is partly vertical and partly inclined [Fig. 19.2 (b)], the resulting water force can be resolved into horizontal component (P_h) and vertical component (P_v). The horizontal component $P_h = \frac{1}{2} \gamma_w H^2$ acts at $\frac{H}{3}$ from the base ; and the vertical component (P_v) is equal to the weight of the water stored in column $ABCA$ and acts at the c.g. of the area.

Similarly, if there is tail water on the downstream side, it will have horizontal and vertical components, as shown in Fig. 19.2. (b).

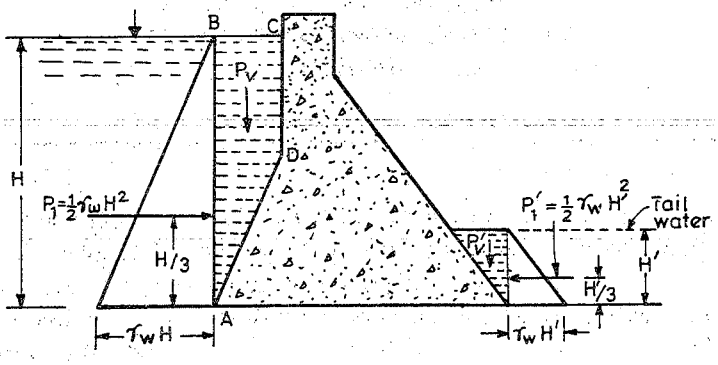


Fig. 19.2 (b)

(2) **Uplift Pressure.** Water seeping through the pores, cracks and fissures of the foundation material, and water seeping through dam body and then to the bottom through the joints between the body of the dam and its foundation at the base ; exert an uplift pressure on the base of the dam. It is the second major external force and must be accounted for in all calculations. Such an uplift force virtually reduces the downward weight of the body of the dam and hence, acts against the dam stability.

The amount of uplift is a matter of research and the present recommendations which are followed, are those suggested by United States Bureau of Reclamation (U.S.B.R.). According to these recommendations, the uplift pressure intensities at the heel and the toe should be taken equal to their respective hydrostatic pressures and joined by a

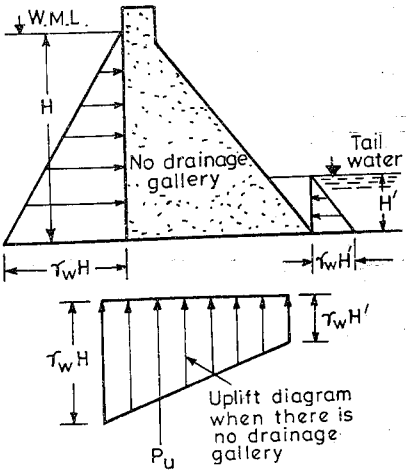


Fig. 19.3 (*a*) Uplift pressure (*U*) diagram, when no drainage gallery is provided.

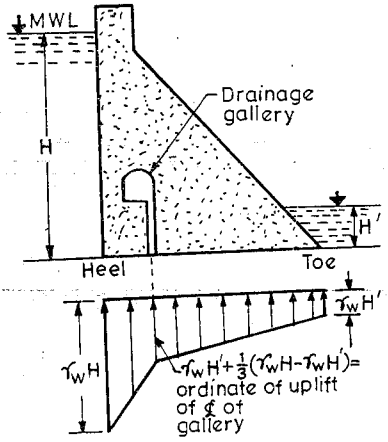


Fig. 19.3 (*b*) Uplift pressure (*U*) diagram, when drainage gallery is provided.

straight line in between, as shown in Fig. 19.3 (*a*). When drainage galleries are provided to relieve the uplift, the recommended uplift at the face of the gallery is equal to the hydrostatic pressure at toe ($\gamma_w \cdot H'$) plus $\frac{1}{3}$rd the difference of the hydrostatic pressures at the heel and the toe ; as shown in Fig. 19.3 (*b*) ; *i.e.* $\left[\gamma_w \cdot H' + \frac{1}{3}(\gamma_w \cdot H - \gamma_w \cdot H')\right]$. It is also assumed that the uplift pressures are not affected by the earthquake forces.

The uplift pressures can be controlled by constructing cut-off walls under the upstream face, by constructing drainage channels between the dam and its foundation, and by pressure grouting the foundation.

(3) **Earthquake Forces.** If the dam to be designed, is to be located in a region which is susceptible to earthquakes, allowance must be made for the stresses generated by the earthquakes.

An earthquake produces waves which are capable of shaking the Earth upon which the dam is resting, in every possible direction.

The effect of an earthquake is, therefore, equivalent to imparting an acceleration to the foundations of the dam in the direction in which the wave is travelling at the moment. Earthquake wave may move in any direction, and for design purposes, it has to be resolved in vertical and horizontal components. Hence, two accelerations, *i.e.* one horizontal acceleration (α_h) and one vertical acceleration (α_v) are induced by an earthquake. The values of these accelerations are generally expressed as percentage of the acceleration due to gravity (g), *i.e.* $\alpha = 0.1\,g$ or $0.2\,g$, etc.

In India, the entire country has been divided into five seismic zones depending upon the severity of the earthquakes. Zone V is the most serious zone and includes Himalayan regions of North India. A map and description of these zones is available in "Physical and Engineering Geology" (1999 edition) by the same author, and can be referred to, in order to obtain an idea of the value of the α which should be chosen for designs. On an average, a value of α equal to 0.1 to 0.15 g is generally sufficient for high dams in seismic zones. A value equal to 0.15 g has been used in Bhakra dam design, and 0.2 g in Ramganga dam design. However, for areas not subjected to extreme earthquakes,

$\alpha_h = 0.1\ g$ and $\alpha_v = 0.05\ g$ may by used. In areas of no earthquakes or very less earthquakes, these forces may be neglected. In extremely seismic regions and in conservative designs, even a value upto 0.3 g may sometimes be adopted.

Effect of vertical acceleration (α_v). A vertical acceleration may either act downward or upward. When it is acting in the upward direction, then the foundation of the dam will be lifted upward and becomes closer to the body of the dam, and thus the effective weight of the dam will increase and hence, the stress developed will increase.

When the vertical acceleration is acting downward, the foundation shall try to move downward away from the dam body ; thus reducing the effective weight and the stability of the dam, and hence is the worst case for designs.

Such acceleration will, therefore, exert an **inertia force** given by

$$\frac{W}{g}\alpha_v \ (i.e.\ \text{force} = \text{Mass} \times \text{Acceleration})$$

where W is the total weight of the dam.

$\therefore$ The net effective weight of the dam $= W - \frac{W}{g} \cdot \alpha_v$.

If $\alpha_v = k_v \cdot g$

[where k_v is the fraction of gravity adopted for vertical acceleration, such as 0.1 or 0.2, etc.].

Then, the net effective weight of the dam

$$= W - \frac{W}{g} \cdot k_v \cdot g = W[1 - k_v].$$

In other words, vertical acceleration reduces the unit weight of the dam material and that of water to $(1 - k_v)$ times their original unit weights.

Effects of horizontal acceleration (α_h). Horizontal acceleration may cause the following two forces :

(*i*) Hydrodynamic pressure ; and

(*ii*) Horizontal inertia force.

Both these forces are discussed below :

(*i*) *Hydrodynamic pressures.* Horizontal acceleration acting towards the reservoir causes a momentary increase in the water pressure, as the foundation and dam accelerate towards the reservoir and the water resists the movement owing to its inertia. The extra pressure exerted by this process is known as *hydrodynamic pressure.*

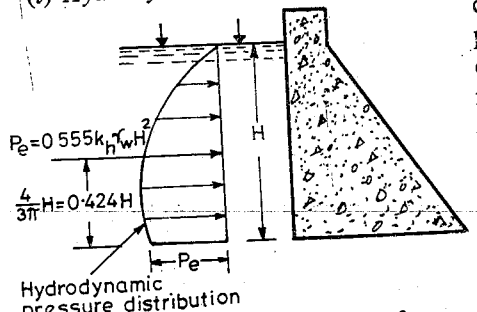


Fig. 19.4. Showing development of Hydrodynamic pressure by a horizontal earthquake moving towards the reservoir. A similar pressure will be developed on d/s tail water when the earthquake is reversed.

According to **Von-Karman**, the amount of this hydrodynamic force (P_e) is given by.

$$P_e = 0.555 \cdot k_h \gamma_w \cdot H^2 \qquad ...(19.1)$$

and it acts at the height of $\frac{4H}{3\pi}$ above the base, as shown in Fig. 19.4.

where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, 0.2 etc.

γ_w = unit wt. of water

Moment of this force about base

$$= M_e = P_e\left(\frac{4H}{3\pi}\right) = \mathbf{0.424\ P_e \cdot H} \qquad ...(19.2)$$

Zanger has given certain big formulas for evaluating the amount of this force and its position, etc. on the vertical as well as on an inclined faces. The results of these big formulas are quite comparable to those given by Von-Karman equation and hence, for average ordinary purposes, the Von-Karman equation (19.1) is sufficient.

Zanger's formula for hydrodynamic force. According to Zanger

$$P_e = 0.726\, p_e \cdot H \qquad ...(19.3)$$

where $p_e = C_m k_h \cdot \gamma_w \cdot H$ ...(19.4)

$$\therefore \quad P_e = 0.726\, C_m \cdot k_h \cdot \gamma_w \cdot H^2 \qquad ...(19.5)$$

where C_m = Maximum value of pressure co-efficient for a given constant slope

$$= 0.735\left(\frac{\theta}{90}^{\circ}\right); \qquad ...(19.6)$$

where θ is the angle in degrees, which the u/s face of the dam makes with the horizontal.

k_h = fraction of gravity adopted for horizontal acceleration (α_h) such as $\alpha_h = k_h \cdot g$

γ_w = unit w.t. of water

The moment of this force about the base is given as :

$$M_e = 0.299\, p_e \cdot H^2 \qquad ...(19.7)$$

$$= 0.299 \frac{P_e}{0.726 \cdot H} \cdot H$$

or $$M_e = 0.412\, P_e \cdot H \qquad ...(19.8)$$

It was further stated, that if the upstream face is partly inclined (Fig. 19.5 *a*), which does not extend to more than half the depth of the reservoir, it can be taken as vertical. If the slope extends to more than half the depth (Fig. 19.5 *b*), the overall slope up to the whole height may be taken as the value of θ in equation (19.6) above.

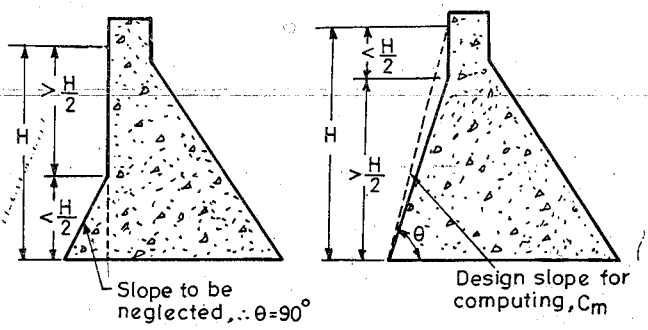


Fig. 19.5

Zanger's formulae are given in details in U.S.B.R. publication on design of small dams, which may be referred to in particular special needs.

(ii) Horizontal Inertia Force. In addition to exerting the hydrodynamic pressure, the horizontal acceleration produces an inertia force into the body of the dam. This force is generated in order to keep the body and the foundation of the dam together as one piece. The direction of the produced force will be opposite to the acceleration imparted by the earthquake.

Since an earthquake may impart either upstream or downstream acceleration, we have to choose the direction of this force in our stability analysis of dam structure, in such a way that it produces most unfavorable effects under the considered conditions.

Say for example, when the reservoir is *full*, this force would produce worst results if it additive to the hydrostatic water pressure, thus acting towards the *downstream* (*i.e.* when upstream earthquake acceleration towards the reservoir is produced). When the reservoir is *empty*, this force would produce worst results, if considered to be acting *upstream* (*i.e.* when earthquake acceleration, moving towards downstream, is produced).

Under reservoir empty conditions, earthquake forces produce effects, which may cause slight tension near the toe ; and hence stability analysis for reservoir empty case may be carried out only on the basis of wt. of the dam by ignoring earthquake forces and keeping the section free from any tension. However, for all precise designs, these forces must be fully considered, as we have done in example 19.2.

The amount of this horizontal inertial force is equal to the product of the mass of the dam and the acceleration.

∴ This horizontal Inertia force

$$= \left(\frac{W}{g}\right)\alpha_h = \frac{W}{g} \cdot k_h \cdot g = W \cdot k_h \qquad \ldots(19.9)$$

(where k_h is the fraction of gravity adopted for horizontal acceleration, such as 0.1, or 0.2, etc.).

This force should be considered to be acting at the centre of gravity of the mass, regardless of the shape of the cross- section, and it acts horizontally downstream in worst cases, for reservoir full case.

(4) **Silt Pressure.** It has been explained under 'Reservoir Sedimentation' in chapter 18 that silt gets deposited against the upstream face of the dam. If h is the height of silt deposited, then the force exerted by this silt in addition to external water pressure, can be represented by Rankine's formula as :

$$P_{silt} = \frac{1}{2} \cdot \gamma_{sub} \cdot h^2 K_a \text{ and it acts at } \frac{h}{3} \text{ from base} \qquad \ldots(19.10)$$

where K_a is the coefficient of active earth pressure of silt $= \dfrac{1 - \sin\phi}{1 + \sin\phi}$ where ϕ is the angle of internal friction of soil, and cohesion is neglected.

γ_{sub} = submerged unit weight of silt material.

h = height of silt deposited.

If the upstream face is inclined, the vertical weight of the silt supported on the slope also acts as a vertical force.

In the absence of any reliable data for the type of silt that is going to be deposited, U.S.B.R. recommendations may be adopted. In these recommendations, deposited silt may be taken as equivalent to a fluid exerting a force with a unit wt. equal to 3.6 kN/m^3 in the horizontal direction and a vertical force with a unit wt. of 9.2 kN/m^3. Hence, the total horizontal force will be $3.6 \frac{h^2}{2} = 1.8\, h^2$ kN/m run, and vertical force will be $9.2 \cdot \frac{h^2}{2} = 4.6\, h^2$ kN/m run.

In most of the gravity-dam designs, the silt pressure is neglected. The basis for neglecting this force is that :

Initially, the silt load is not present, and by the time it becomes significant, it gets consolidated to some extent and, therefore, acts less like a fluid. Moreover, silt deposited in the reservoir is somewhat impervious and, therefore, will help to minimise the uplift under the dam.

(5) **Wave Pressure.** Waves are generated on the surface of the reservoir by the blowing winds, which causes a pressure towards the downstream side. Wave pressure depends upon the wave height. Wave height may be given by the equation,

$$h_w = 0.032 \sqrt{V \cdot F} + 0.763 - 0.271\, (F)^{3/4} \textbf{ for F < 32 km,} \text{ and} \quad ...(19.11)$$

$$h_w = 0.032 \sqrt{V \cdot F} \textbf{ for F > 32 km} \quad ...(19.12)$$

where h_w = height of water from top of crest to bottom of trough in metres.

V = wind velocity in km/hr.

F = Fetch or straight length of water expanse in km.

The maximum pressure intensity due to wave action may by given by

$p_w = 2.4\, \gamma_w \cdot h_w$ and acts at $\frac{h_w}{2}$ metres above the still water surface.

The pressure distribution may be assumed to be triangular, of height $\frac{5h_w}{3}$, as shown in Fig. 19.6.

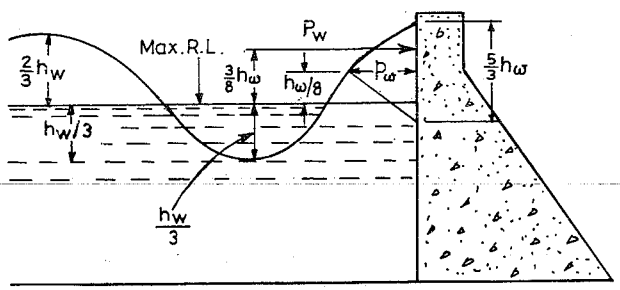


Fig. 19.6

Hence, the total force due to wave action (P_w)

$$= \frac{1}{2} (2.4\, \gamma_w \cdot h_w) \cdot \frac{5}{3} \cdot h_w$$

or $$P_w = 2 \cdot \gamma_w \cdot h_w^2 = 2 \times 9.81\, h_w^2 \text{ kN/m}$$
$$= 19.62\, h_w^2 \text{ kN/m} \quad \ldots(19.14)$$

This force acts at a distance $\frac{3}{8} h_w$ above the reservoir surface.

(6) **Ice Pressure.** The ice which may be formed on the water surface of the reservoir in cold countries, may sometimes melt and expand. The dam face has then to resist the thrust exerted by the expanding ice. This force acts linearly along the length of the dam and at the reservoir level. The magnitude of this force varies from 250 to 1500 kN/m^2 depending upon temperature variations. On an average, a value of 500 kN/m^2 may be allowed under ordinary conditions.

(7) **Weight of the Dam.** The weight of the dam body and its foundation is the major resisting force. In two dimensional analysis of a gravity dam, a unit length of the dam is considered. The cross-section can then be divided into rectangles and triangles. The weight of each along with their *c.gs.*, can be determined. The resultant of all these downward forces will represent the total weight of the dam acting at the *e.g.* of the dam.

Combination of forces for Designs. The design of a gravity dam should be checked for two cases, *i.e.* (*i*) when Reservoir is full ; and (*ii*) when Reservoir is empty.

(*i*) **Case I.** Reservoir full case :

When reservoir is full, the major forces acting are : weight of the dam, external water pressure, uplift pressure, and earthquake forces in serious seismic zones. The minor forces are : silt pressure, ice pressure and wave pressure. For the most conservative designs, and from purely theoretical point of view, one can say that a situation may arise when all the forces may act together. But such a situation will never arise and hence, all the forces are not generally taken together. U.S.B.R. has classified the 'normal load combinations' and 'extreme load combination, as given below :

(*a*) *Normal Load Combinations*

(*i*) Water pressure upto normal pool level, normal uplift, silt pressure and ice pressure. This class of loading is taken when ice force is serious.

(*ii*) Water pressure upto normal pool level, normal uplift, earthquake forces, and silt pressure.

(*iii*) Water pressure upto maximum reservoir level (maximum pool level), normal uplift, and silt pressure.

(*b*) *Extreme Load Combinations*

(*i*) Water pressure due to maximum pool level, extreme uplift pressure without any reduction due to drainage and silt pressure.

Case II. Reservoir empty case :

(*i*) Empty reservoir without earthquake forces to be computed for determining bending diagrams, etc. for reinforcement design, for grouting studies or other purposes.

(*ii*) Empty reservoir with a horizontal earthquake force produced towards the upstream has to be checked for non- development of tension at toe.

19.4. Modes of Failure and Criteria for Structural Stability of Gravity Dams

A gravity dam may fail in the following ways :

(1) By overturning (or rotation) about the toe.

(2) By crushing.

(3) By development of tension, causing ultimate failure by crushing.

(4) By shear failure called sliding.

The failure may occur at the foundation plane (*i.e.* at the base of the dam) or at any other plane at higher level.

(1) **Over-turning.** If the resultant of all the forces acting on a dam at any of its sections, passes outside the toe, the dam shall rotate and overturn about the toe. Practically, such a condition shall not arise, as the dam will fail much earlier by compression. The ratio of the righting moments about toe (anti clockwise) to the over turning moments about toe (clock-wise) is called the factor of safety against overturning. Its value, generally varies between 2 to 3.

(2) **Compression or crushing.** A dam may fail by the failure of its materials, *i.e.* the compressive stresses produced may exceed the allowable stresses, and the dam-material may get crushed. The vertical direct stress distribution at the base is given by the equation :

$$p = \text{Direct stress} + \text{Bending stress.}$$

$$\therefore \quad p_{\substack{max \\ min}} = \frac{\Sigma V}{B} \pm \frac{M}{I} y = \frac{\Sigma V}{B} \pm \frac{\Sigma V \cdot e}{B^2/6} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

or

$$p_{\substack{max \\ min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad ...(19.15)$$

where e = Eccentricity of the resultant force from the centre of the base.

ΣV = Total vertical force.

B = Base width.

Note. Resultant is nearer the toe and hence, maximum compressive stress is produced at the toe (Reservoir full case)

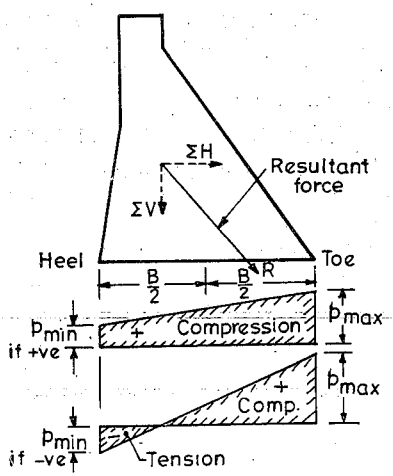


Fig. 19.7. (*a*) Vertical Stress Distribution for Reservoir Full case.

The maximum stress, *i.e.* p_{max}, will be produced on the end which is nearer to the resultant, as shown in Fig. 19.7 (*a*) and (*b*).

Note. The resultant is nearer the heel and hence, maximum compressive stress (+ve stress) is produced at the heel (Reservoir empty with horizontal earthquake wave moving away from reservoir-case).

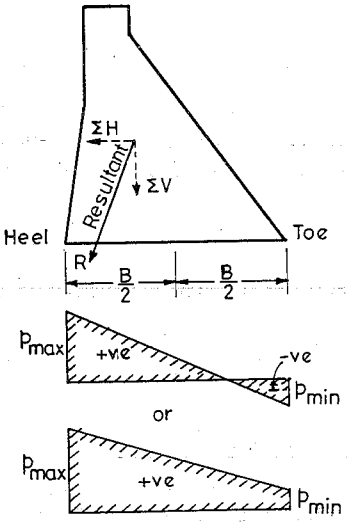


Fig. 19.7. (*b*) Vertical Stress Distribution for Reservoir Empty case.

If p_{min} comes out to be negative, it means that tension shall be produced at the appropriate end.

If p_{min} exceeds the allowable compressive stress of dam material [generally taken as 3000 kN/m^2 (30 kg/cm^2) for concrete]; the dam may crush and fail by crushing.

(3) **Tension.** Masonry and concrete gravity dams are usually designed in such a way that no tension is developed anywhere, because these materials cannot withstand sustained tensile stresses. If subjected to such stresses, these materials may finally crack. However, for achieving economy in designs of very high gravity dams, certain amount of tension may be permitted under severest loading condition. This may be permitted because of the fact that such worst loading conditions shall occur only momentarily for a little time and would neither last long nor occur frequently. The maximum permissible tensile stress for high concrete gravity dams, under worst leadings, may be taken as 500 kN/m^2 (5 kg/cm^2).

Effect produced by tension cracks. In a dam, when such a tension crack develops, say at the heel, crack width (or strictly speaking crack-area) looses contact with the bottom foundations, and thus, becomes ineffective.

Hence, the effective width B (considering unit length) of the dam base will be reduced. This will increase p_{max} at the toe.

Moreover, the uplift pressure diagram gets modified due to crack formation, as shown in Fig. 19.8, resulting in an increase in the uplift. Since the uplift increases and the net effective downward force reduces, the resultant will shift more towards the toe and thus further increasing the compressive stress at the toe and further lengthening the crack due to further tension development. The process continues ; the effective base width goes on reducing and compressive stress at the toe goes on increasing ; finally leading to the failure of the toe by direct compression. *Hence, a tension crack by itself does not fail the structure, but it leads to the failure of the structure by producing excessive compressive stresses.*

ABC = old uplift diagram
$A'B'C'$ = New uplift diagram after the crack AA', has devloped.

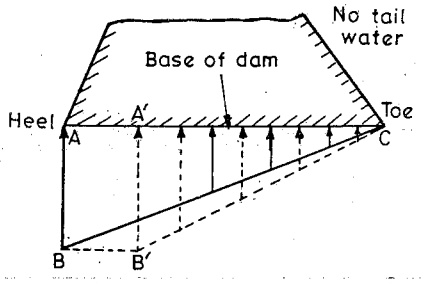


Fig. 19.8

In order to ensure that no tension is developed anywhere, we must ensure that p_{min} is at the most equal to zero.

Since
$$p_{\substack{max\\min}} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad ...(19.15)$$

$$p_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right]$$

If $p_{min} = 0$,

$$\frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] = 0$$

or
$$1 - \frac{6e}{B} = 0$$

or
$$e = \frac{B}{6}.$$

Hence, maximum value of eccentricity that can be permitted on either side of the centre is equal to $\frac{B}{6}$; which leads to the famous statement : *the resultant must lie within the middle third.*

(4) **Sliding.** Sliding (or shear failure) will occur when the net horizontal force above any plane in the dam or at the base of the dame exceeds the frictional resistance developed at that level.

The friction developed between two surfaces is equal to $\mu\Sigma V$., (Fig. 19.9) where ΣV is the algebraic sum of all the vertical forces whether upward or downward, and μ is the coefficient of friction between the two surfaces. In order that no sliding takes place, the external horizontal forces (ΣH) must be less than the shear resistance $\mu \cdot \Sigma V$.

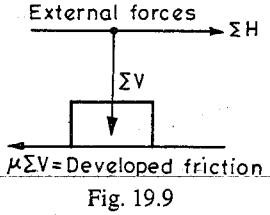


Fig. 19.9

or
$$\Sigma H < \mu\Sigma V.$$

or
$$\frac{\mu\Sigma V}{\Sigma H} > 1$$

$\frac{\mu \cdot \Sigma V}{\Sigma H}$ represents nothing but the factor of safety against sliding, which must be greater than unity.

$\therefore$ F.S.S. (Factor of safety against sliding) = $\frac{\mu \cdot \Sigma V}{\Sigma H}$.

In low dams, the safety against sliding should be checked only for friction, but in high dams, for economical precise designs, the shear strength of the joint, which is an additional shear resistance, must also be considered. If this shear resistance of the joint is also considered, then the equation for factor of safety against sliding which is measured by shear friction factor (S.F.F.) becomes

$$\text{S.F.F.} = \frac{\mu \Sigma V + B \cdot q}{\Sigma H} \qquad ...(19.16)$$

where B = width of the dam at the joint,

q = Average shear strength of the joint which varies from about 1400 kN/m^2 (14 kg/cm^2) for poor rocks to about 4000 kN/m^2 (40 kg/cm^2) for good rocks.

The value of μ generally varies from 0.65 to 0.75.

Attempts are always made to increase this shear strength (q) at the base and at other joints. For this purpose, foundation is stepped at the base, as shown in Fig. 19.10 and measures are taken to ensure a better bond between the dam base and the rock-foundation.

During the construction of a dam, horizontal joints have to be left as shown in Fig. 19.10. The shear strength of these joints should be made as good as possible by ensuring better bond between the two surfaces. For this purpose, the lower surface must be thoroughly cleaned and a layer of neat cement or rich cement mortar should be spread before pouring the standard concrete mix for the upper layer. If these precautions of quality control are not adhered to in the filed, the assumption made in accounting for this shear strength in the design, will not be justified. That is why, for small dams, where quality control is less, this shear strength of the joint is not taken into account at all, while determining the shear friction factor or factor of safety against sliding.

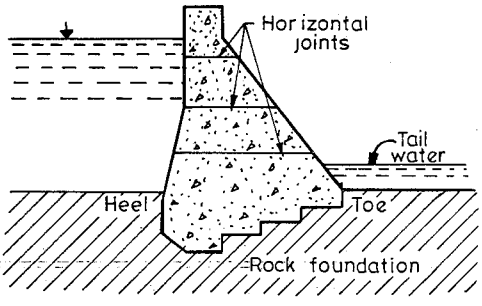


Fig. 19.10

19.4.1. Principal and Shear Stresses. The vertical stress intensity, p_{max} or p_{min} determined from the equation (19.15) is not the maximum direct stress produced anywhere in the dam. The maximum normal stress will, in fact, be the major principal stress that will be generated on the major principal plane. When the reservoir is full, the vertical direct stress [given by equation (19.15), and represented by p_v in future] is maximum at the toe as the resultant is nearer to the toe. To study the principal stresses that will develop near the toe, let us consider a small element ABC [See Fig. 19.11 (a) and (b)] near the toe of

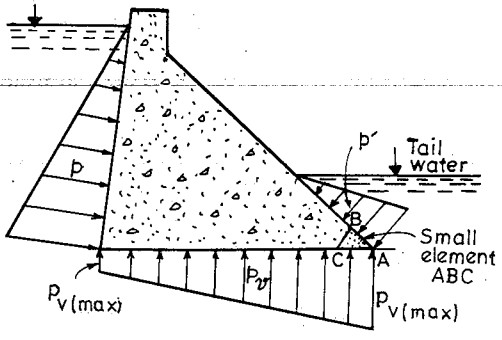


Fig. 19.11 (a)

the dam. The element is so small that the stress intensities may be assumed to be uniform on its faces.

Let the downstream face of the dam be inclined at an angle α to the vertical.

This face of the dam will act as a principal plane because the water pressure p' acts at right angles to the face, and also there is no shear stress acting on this plane. Since the principal planes are at right angles to each other ; the plane BC drawn at right angles to the face AB will be the second principal plane. Let the stress acting on this plane be σ.

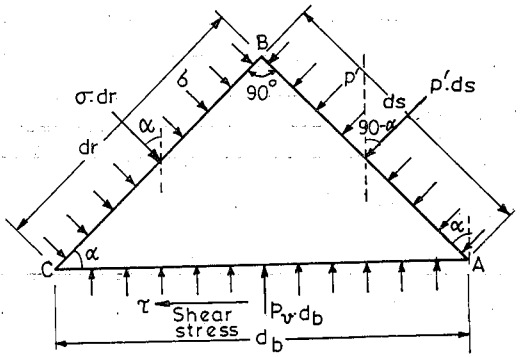


Fig. 19.11. (*b*) Enlarged view of small element *ABC* of Fig. 19.11 (*a*).

Let ds, dr and db be the lengths of AB, BC and CA respectively. p' is the intensity of water pressure on face AB and p_v is the intensity of vertical pressure on face AC, and σ is the intensity of normal stress (principal stress) on face BC. Considering unit length of the dam, the forces acting on the faces AB, BC and CA are $p' \, ds$, $\sigma \, dr$ and $p_v \cdot db$ respectively.

Resolving all the forces in the vertical direction, we get

$$p' \cdot ds \cdot \sin\alpha + \sigma \cdot dr \cdot \cos\alpha = p_v \cdot db,$$

Now $$\frac{ds}{db} = \sin\alpha, \quad \text{or } ds = db \cdot \sin\alpha.$$

$$\frac{dr}{db} = \cos\alpha, \quad \text{or } dr = db \cdot \cos\alpha.$$

$\therefore$ $$p' \cdot (db \cdot \sin\alpha) \cdot \sin\alpha + \sigma \cdot (db \cdot \cos\alpha) \cos\alpha = p_v \cdot db$$

or $$p' \cdot \sin^2 a + \sigma \cdot \cos^2\alpha = p_v.$$

or $$\sigma = \frac{p_v - p' \cdot \sin^2\alpha}{\cos^2\alpha}$$

or $$\boxed{\sigma = p_v \cdot \sec^2\alpha - p' \tan^2\alpha} \qquad ...(19.17)$$

For σ to be maximum, p' should be zero, *i.e.* when there is no tail water ; then in such a case

$$\sigma = p_v \cdot \sec^2\alpha \qquad ...[19.17\ (a)]$$

Since $\sec^2\alpha$ is always more than 1, it follows, that σ will be more than p_v. *This value of normal stress, which is the maximum produced anywhere in the body of the dam, must be calculated and should not be allowed to exceed the maximum allowable compressive stress of dam material.*

If the hydrodynamic pressure (p_e') exerted by the tail water during an earthquake moving towards the reservoir is also considered, then the net pressure on the face AB will be $(p' - p_e')$, because the effect of this earthquake will be to reduce the tail water pressure.

The principal stress (σ) can then be given by

or $$\boxed{\sigma_{at\ toe} = p_v \cdot \sec^2 \alpha - (p' - p_e') \tan^2 \alpha} \quad ...(19.18)$$

The equation for σ, derived above for the element at the toe is also applicable to the element at the heel. The equation at the heel is, therefore, given as :

$$\sigma_1 = \sigma_{at\ heel} = p_v \cdot \sec^2 \phi - (p + p_e) \tan^2 \phi \quad ...(19.19)$$

where ϕ is the angle which the u/s face makes with vertical.

But at the heel, the pressure of water p is always more than σ, and hence, p will be the minor principal stress at the heel.

Shear stress on the horizontal plane near the toe. A shear stress τ will act on the face *CA* on which the vertical stress is acting. Resolving all the forces [Fig. 19.11 (*b*)] in the horizontal direction, we get

$$\sigma \cdot dr \sin \alpha - p' \cdot ds \cdot \cos \alpha = \tau_0 \cdot db$$

or $$\sigma \cdot (db \cdot \cos \alpha) \sin \alpha - p' (db \cdot \sin \alpha) \cos \alpha = \tau_0 \cdot db$$

or $$\sigma \cdot \sin \alpha \cos \alpha - p' \sin \alpha \cos \alpha = \tau_0$$

or $$\tau_0 = (\sigma - p') \sin \alpha \cos \alpha$$

Substituting the value of σ from equation (19.17), we get

$$\tau_0 = \left[\overline{p_v \sec^2 \alpha - p' \tan^2 \alpha} - p' \right] \sin \alpha \cos \alpha$$

$$\tau_0 = \left[p_v \sec^2 \alpha - p' (1 + \tan^2 \alpha) \right] \sin \alpha \cos \alpha = \left[(p_v - p') \sec^2 \alpha \right] \sin \alpha \cos \alpha$$

or $$= [(p_v - p') \sec^2 \alpha \cdot \sin \alpha \cdot \cos \alpha]$$

or $$\boxed{\tau = (p_v - p') \tan \alpha} \quad ...(19.20)$$

Neglecting tail water, shear stress is given by

$$\tau_0 = p_v \cdot \tan \alpha \quad ...[19.20\ (a)]$$

If the effect of hydrodynamic pressure produced by an earthquake moving towards the reservoir, is also considered, the equation for shear stress on a horizontal plane near the toe becomes,

$$\tau_0 = \left[p_v - (p' - p_e') \right] \tan \alpha \quad ...(19.21)$$

Similarly, shear stress at heel

$$= \tau_{0\ (heel)} = \left[p_v - (p + p_e) \right] \tan \phi$$

–ve sign shows that the direction is reversed.

Example 19.1. *Fig. 19.12 (a) shows the section of a gravity dam (non-overflow portion) built of concrete.*

Calculate (neglecting earthquake effects)

(*i*) *The maximum vertical stresses at the heel and toe of the dam.*

(*ii*) *The major principal stress at the toe of the dam.*

(*iii*) *The intensity of shear stress on a horizontal plane near the toe.*

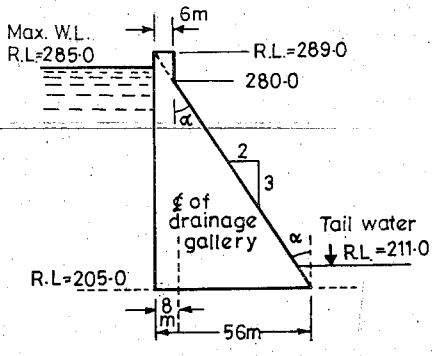


Fig. 19.12 (*a*)

Assume weight of concrete = 23.5 kN/m^3; and unit length of dam. Allowable stress in concrete may be taken 2500 kN/m^2

Solution. Assuming $\gamma_w = 9.81\ \text{kN/m}^3$; the various forces acting on the dam are drawn in Fig. 19.12 (*b*).

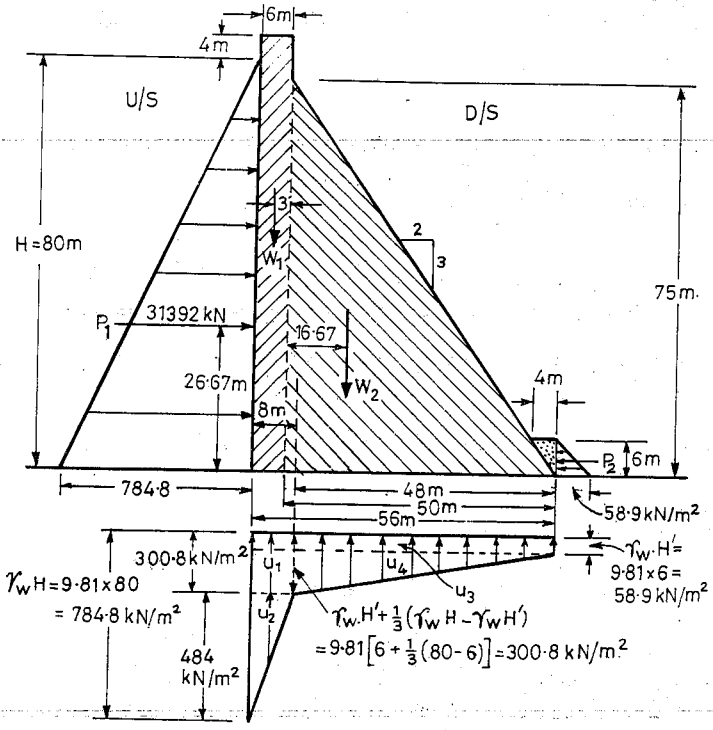


Fig. 19.12 (*b*)

Consider 1 m length of the dam.

The various forces and their moments about the toe are then calculated and tabulated in Table 19.1. From this table, we have

Distance of resultant from the toe

$$(\bar{x}) = \frac{\Sigma M}{\Sigma V}$$

$$= \frac{7,77,639\ \text{kN} \cdot m}{43050\ \text{kN}} = 18.06\ \text{m}$$

$$\text{Eccentricity} = e = \frac{56}{2} - 18.06 = 28 - 18.06 = 9.94\ \text{m}$$

Vertical stress p_v is given as :

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \qquad p_v = \frac{43,050\ \text{kN}}{56\ \text{m}}\left[1 \pm \frac{6 \times 9.94}{56}\right] = 768.8\,(1 \pm 1.065)$$

Table 19.1

Name of the force	*Designation if given*	*Magnitude in kN*	*Lever arm in m*	*Moments about toe in kN.m*
Vertical forces				
Downward weight of the dam	W_1	(+) $84 \times 6 \times 1 \times 23.5 = 11{,}844$	53.0	(+) 6,27,732
	W_2	(+) $\frac{1}{2} \times 50 \times 75 \times 1 \times 23.5 = 44{,}063$	33.33	(+) 14,68,620
Weight of water supported on d/s face	—	(+) $\frac{1}{2} \times 4 \times 6 \times 1 \times 9.81 = 118$	1.33	(+) 157
		$\Sigma V_1 = 56{,}025$		$\Sigma M_1 = (+)\ 20{,}96{,}509$
Uplift pressures	U_1	(−) $300.8 \times 8 \times 1 = 2406$	52.0	(−) 1,25,112
—	U_2	(−) $\frac{1}{2} \times 484 \times 8 \times 1 = 1936$	53.33	(−) 1,03,247
	U_3	(−) $58.9 \times 48 \times 1 = 2827$	24.0	(−) 67,848
	U_4	(−) $\frac{1}{2} \times 241.9 \times 48 \times 1 = 5806$	32.0	(−) 1,85,792
		$\Sigma V_2 = (-)\ 12{,}975$		$\Sigma M_2 = (-)\ 4{,}81{,}999$
		$\Sigma V = 56{,}025 - 12{,}975 = \mathbf{43050}$		
Horizonal *Water pressure*				
On u/s face	P	$\frac{1}{2} \times 784.8 \times 80 \times 1 = 31{,}392$	26.67	(−) 8,37,225
On d/s face	P'	(−) $\frac{1}{2} \times 58.9 \times 6 = (-)\ 177$	2.0	(+) 354
		ΣH (towards downstream) = 31,215		$\Sigma = (-)\ 8{,}36{,}871$
ΣM = Net (+) moment = 20,96,509 − 4,81,999 − 8,36,871 = 7,77,639 kN-m				

$\therefore$ Max. vertical stress = p_{max} at toe = $768.8 \times 2.065 = \mathbf{1587.6\ kN/m^2}$

$\therefore$ Min. vertical stress = p_{min} at heel = $768.8 \times (-)\ 0.065 = (-)\ \mathbf{49.97\ kN/m^2}$ **Ans.**

(*ii*) Major principal stress at toe (σ) is given by Eq. (19.17) as :

$$\sigma = p_{v\,(toe)} \sec^2 \alpha - p' \cdot \tan^2 \alpha$$

here $p_{v\,(toe)} = 1587.6\ \text{kN/m}^2$

$p' = 58.9\ \text{kN/m}^2$

$$\tan \alpha = \frac{2}{3}$$

$$\sec \alpha^2 = 1 + \tan^2 \alpha = 1 + \frac{4}{9} = \frac{13}{9}$$

$\therefore$ $$\sigma = 1587.6 \times \frac{13}{9} = 58.9 \times \frac{4}{9}$$

$$= \mathbf{2267\ kN/m^2 < 2500\ kN/m^2\ (OK)} \quad \textbf{Ans.}$$

(*iii*) Intensity of shear stress on a horizontal plane near toe is given by equation (19.20)

$\therefore$ $$\tau_0 = \left[p_{v(toe)} - p'\right] \tan \alpha$$

$$= 1587.6 - 58.9)\ \tfrac{2}{3} = \mathbf{1019.1\ kN/m^2}. \quad \textbf{Ans.}$$

STABILITY ANALYSIS

The stability of a gravity dam can be approximately and easily analysed by two dimensional gravity method and can be precisely analysed by three dimensional methods such as slab analogy method, trial load twist method, or by experimental studies on models. Two dimensional gravity method is discussed below :

19.5. Gravity Method or Two Dimensional Stability Analysis

The preliminary analysis of all gravity dams can be made easily by isolating a typical cross-section of the dam of a unit width. This section is assumed to behave independently of the adjoining sections. In other words, the dam is considered to be made up of a number of cantilevers of unit width each, which act independently of each other. This assumption of independent functioning of each section, disregards the beam action in the dam as a whole.

If the vertical transverse joints of the dam are not grouted or keyed together, this assumption is nearly true. Hence, for wide *U*-shaped valleys, where transverse joints are not generally grouted, this assumption is nearly satisfied. But for narrow V- shaped valleys, where the transverse joints are generally keyed together and the entire length of the dam acts monolithically as a single body, this assumption may involve appreciable errors. In such cases, preliminary designs may be done by gravity method and precise final designs may be carried out by any of the available three dimensional methods.

The description of the three dimensional methods is beyond the scope of this book, and only the two dimensional analysis has been used for the design of gravity dams in this chapter.

Assumptions. The various assumptions made in the two dimensional designs of gravity dams are summarised below :

(*i*) The dam is considered to be composed of a number of cantilevers, each of which is 1 m thick and each of which acts independent of the other.

(*ii*) No loads are transferred to the abutments by beam action.

(*iii*) The foundation and the dam behave as a single unit ; the joint being perfect.

(*iv*) The materials in the foundation and body of the dam are isotropic and homogeneous.

(*v*) The stresses developed in the foundation and body of the dam are within elastic limits.

(*vi*) No movements of the foundations are caused due to transference of loads.

(*vii*) Small openings made in the body of the dam do not affect the general distribution of stresses and they only produce local effects as per St. Venant's principle.

Procedure. Two dimensional analysis can be carried out analytically or graphically.

(*a*) *Analytical Method.* The stability of the dam can be analysed in the following steps :

(*i*) Consider unit length of the dam.

(*ii*) Work out the magnitude and directions of all the vertical forces acting on the dam and their algebraic sum, *i.e.* ΣV.

(*iii*) Similarly, work out all the horizontal forces and their algebraic sum, *i.e.* ΣH.

(*iv*) Determine the lever arm of all these forces about the toe.

(*v*) Determine the moments of all these forces about the toe and find out the algebraic sum of all those moments, *i.e.* ΣM.

(*vi*) Find out the location of the resultant force by determining its distance from the toe,

$$\bar{x} = \frac{\Sigma M}{\Sigma V}.$$

(*xii*) Find out the eccentricity (e) of the resultant (R) using $e = \frac{B}{2} - \bar{x}$. It must be less than $B/6$ in order to ensure that no tension is developed anywhere in the dam.

(*viii*) Determine the vertical stresses at the toe and heel using Eq. (19.15), *i.e.*

$$p_v = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \qquad ...(19.15)$$

Sometimes stresses are found by ignoring uplift.

(*ix*) Determine the maximum normal stresses, *i.e.* principal stresses at the toe and the heel using equations (19.18) to (19.20). They should not exceed the maximum allowable values. The crushing strength of concrete varies between 1500 to 3000 kN/m^2 depenting upon its grade M15 to M3O.

(*x*) Determine the factor of safety against overturning as equal to $\frac{\Sigma \text{ Stabilising moment (+)}}{\Sigma \text{ Disturbing moment (−)}}$; +ve sign is used for anti-clockwise moments and −ve sign is used for clockwise moments.

(*xi*) Determine the factor of safety against sliding, using Sliding factor $= \frac{\mu \Sigma V}{\Sigma H}$.

$$\text{Shear friction factor (S.F.F.)} = \frac{\mu \Sigma V + bq}{\Sigma H}.$$

Sliding factor must be greater than unity and S.F.F. must be greater than 3 to 5. The analysis should be carried out for reservoir full case as well as for reservoir empty case. The entire procedure has been illustrated in example 19.2.

(*b*) *Graphical method.* In the graphical method, the entire dam section is divided into a number of horizontal sections at some suitable intervals, particularly at the places where the slope changes, as shown in Fig. 19.13. For each section, the sum of the vertical forces (ΣV) and the sum of all the horizontal forces (ΣH) acting above that particular section, are worked out and the resultant force (R) is drawn, graphi-

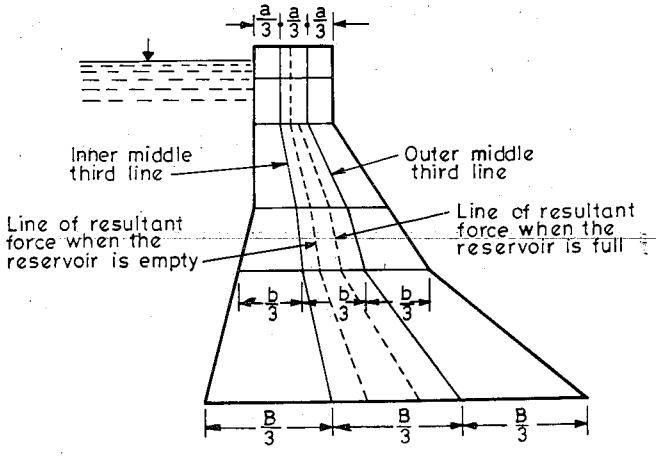


Fig. 19.13

cally. This is done for each section and a line joining all the points where the individual resultants cut the individual sections, is drawn. This line represents the resultant force and should lie within the middle third, for no tension to develop. The procedure should be carried out for reservoir full case as well as for reservoir empty case. The resultant in both cases must show non-development of tension in the dam body.

19.6. Elementary Profile of a Gravity Dam

The elementary profile of a dam, subjected only to the external water pressure on the upstream side, will be a right-angled triangle, having zero width at the water level and a base width (B) at bottom *i.e.*, the point where the maximum hydrostatic water pressure acts. In other words, the shape of such a profile is similar to the shape of the hydrostatic pressure distribution (Fig. 19.14).

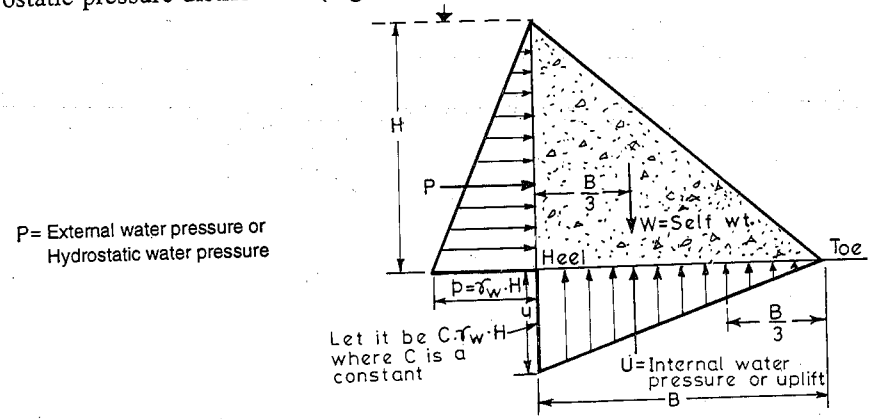


Fig. 19.14

When the reservoir is empty, the only single force acting on it is the self-weight (W) of the dam and it acts at a distance $B/3$ from the heel. This is the maximum possible innermost position of the resultant for no tension to develop. Hence, such a line of action of W is the most ideal, as it gives the maximum possible stabilising moment about the toe without causing tension at toe, when the reservoir is empty. *The vertical stress distribution at the base, when the reservoir is empty,* is given as :

$$P_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Here $\Sigma V = W$

$$e = \frac{B}{6}.$$

$$\therefore \quad p_{max/min} = \frac{W}{B}\left[1 \pm \frac{6 \cdot B}{B \cdot 6}\right]$$

or $$p_{max} = \frac{2W}{B}$$

and $$p_{min} = 0.$$

Hence, the maximum vertical stress equal to $\frac{2W}{B}$ will act at the heel ($\because$ the resultant is nearer the heel) and the vertical stress at toe will be zero.

When the reservoir is full, the base width is governed by :

(*i*) The resultant of all the forces, *i.e.* P, W and U (Fig. 19.14) passes through the outer most middle third point (*i.e.* lower middle third point).

(*ii*) The dam is safe in sliding.

(*i*) For the 1st condition to be satisfied, we proceed as follows : Taking moments of all the forces (Fig. 19.14) about the lower middle third point (*i.e.* the point through which resultant is passing), we get

$$W\left(\frac{B}{3}\right) - U\left(\frac{B}{3}\right) - P\frac{H}{3} = R \times 0$$

or $$(W - U)\frac{B}{3} - P\frac{H}{3} = 0$$

But $$W = \frac{1}{2} \times B \times H \times 1 \times S_c \times \gamma_w$$

where S_c = Sp. gravity of concrete, *i.e.* that of the material of the dam.

γ_w = unit wt. of water = 9.81 kN/m^3

Let the uplift at the heel be $C \cdot \gamma_w \cdot H.$, where C is a constant which according to U.S.B.R. recommendation is taken equal to 1.0 in calculation and will be equal to zero when no uplift is considered.

$\therefore$ $$U = \left(\frac{1}{2} C \cdot \gamma_w \cdot H\right) B$$

and $$P = \frac{1}{2} \gamma_w \cdot H \cdot H = \frac{\gamma_w H^2}{2}$$

$\therefore$ Equation $(W - U)\frac{B}{3} - P\frac{H}{3} = 0$, becomes

$$\left[\frac{1}{2} B \cdot H \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B \cdot\right]\frac{B}{3} - \frac{\gamma_w H^2}{2} \cdot \frac{H}{3} = 0$$

or $$\frac{B}{3} \times \frac{1}{2} B \cdot H \cdot \gamma_w \cdot [S_c - C] = \frac{\gamma_w H^3}{6}$$

or $$B^2 (S_c - C) = H^2$$

or $$\boxed{B = \frac{H}{\sqrt{S_c - C}}} \qquad ...(19.22)$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S_c - C}}$, no tension will be developed at the heel with full reservoir,

when $C = 1$

$$B = \frac{H}{\sqrt{S_c - 1}} \qquad ...[19.22\ (a)]$$

If uplift is not considered, $B = \frac{H}{\sqrt{S_c}}$ ($\because$ $C = 0$) ...[19.22 (*b*)]

(*ii*) For the II condition (*i.e.* dam is safe in sliding) to be satisfied ; the frictional resistance $\mu \Sigma V$ or $\mu (W - U)$ should be equal to or more than the horizontal forces $\Sigma H = P$.

or $$\mu (W - U) \geq P$$

or $$\mu \left(\frac{1}{2} BH \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B. \right) \geq \frac{\gamma_w H^2}{2}$$

or $$\mu (S_c - C) \frac{1}{2} \cdot B \cdot H \cdot \gamma_w \geq \frac{\gamma_w H^2}{2}$$

or $$\mu (S_c - C) B \geq H$$

or $$B \geq \frac{H}{\mu (S_c - C)}$$

Under limiting condition

or $$\boxed{B = \frac{H}{\mu (S_c - C)}} \quad ...(19.23)$$

If $C = 1$ $$B = \frac{H}{\mu (S_c - 1)} \quad ...[19.23(a)]$$

If $C = 0$, *i.e.* no uplift is considered, then

$$B \geq \frac{H}{\mu S_c} \quad ...[19.23(b)]$$

The value of B chosen should be greater of the two values given by Equations (19.22) and (19.23).

Using $S_c = 2.4$ and $\mu = 0.7$ and $C = 0$, we get

$$B \text{ (by Equation 19.22)} = \frac{H}{\sqrt{2.4 - 0}} = \frac{H}{\sqrt{2.4}}$$

$$B \text{ (by Equation 19.23)} = \frac{H}{0.7 \times (2.4 - 0)} = \frac{H}{0.7 \times 2.4} = \frac{H}{1.68}$$

But $\frac{H}{1.68}$ is less than $\frac{H}{\sqrt{2.4}}$

$\therefore$ For all practical purposes, the base width may be taken as $\frac{H}{\sqrt{S_c}}$

The vertical stress distribution when reservoir is full is given as :

$$p_{max/min} = \frac{\Sigma V}{B} \left[1 \pm \frac{6e}{B} \right]$$

where $$\Sigma V = W - U$$

$$= \left(\frac{1}{2} B \cdot H \cdot 1 \cdot S_c \cdot \gamma_w - \frac{1}{2} C \cdot \gamma_w \cdot H \cdot B \right)$$

$$= \frac{1}{2} B \cdot \gamma_w \cdot H \cdot [S_c - C]$$

$$e = \frac{B}{6}$$

$$\therefore \quad p_{max/min} = \frac{\frac{1}{2} \cdot B \cdot \gamma_w \cdot H (S_c - C)}{B} \left[1 \pm \frac{6B}{6B}\right]$$

maximum stress will occur at toe, because the resultant is near the toe.

$$\therefore \quad p_{max} \text{ at toe} = \frac{1}{2} \gamma_w \cdot H (S_c - C)\, 2.0 = \gamma_w H (S_c - C)$$

$$\boxed{p_v \text{ at toe} = \gamma_w H (S_c - C)} \qquad \text{...(19.24)}$$

$$P_{\min} \text{ at heel} = 0.$$

The principal stress near the toe (σ) which is the maximum normal stress in the dam, is given by Equation (19.17)

$$\sigma = p_v \sec^2 \alpha - p' \tan^2 \alpha$$

when there is no tail water *i.e.*, $p' = 0$

$$\sigma = p_v \sec^2 \alpha$$

σ at toe, with full reservoir in elementary profile

$$= \gamma_w H (S_c - C) \sec^2 \alpha$$

$$= \gamma_w H (S_c - C) [1 + \tan^2 \alpha]$$

$$= \gamma_w H (S_c - C) \left[1 + \frac{B^2}{H^2}\right]$$

But $B = \dfrac{H}{\sqrt{S_c - C}}$ from Eq. (19.22)

or $\dfrac{B^2}{H^2} = \dfrac{1}{S_c - C}$

$$\therefore \quad \sigma = \gamma_w H (S_c - C) \left[1 + \frac{1}{S_c - C}\right]$$

or

$$\boxed{\sigma = \gamma_w H (S_c - C + 1)} \qquad \text{...(19.25)}$$

when $C = 1$, $S_c = 2.4$

$$\sigma = \gamma_w H \left(\frac{2.4 - 1 + 1}{2.4 - 1}\right) = \frac{2.4}{1.4} \gamma_w H$$

$$= \mathbf{1.71\, \gamma_w H.}$$

The shear stress τ_0 at a horizontal plane near the toe is given by the equation (19.20) as :

$$\tau_0 = (p_v - p') \tan \alpha$$

If $p' = 0$

$$\tau_0 = p_v \tan \alpha$$

But $p_v = \gamma_w H (S_c - C)$ from Eq. (19.24)

$$\therefore \quad \tau_0 = \gamma_w H (S_c - C) \tan \alpha$$

or

$$\tau_0 = \gamma_w \cdot H (S_c - C) \frac{B}{H}$$

$$= \gamma_w H (S_c - C) \frac{1}{\sqrt{S_c - C}}$$

or $$\boxed{\tau_0 = \gamma_w H \sqrt{S_c - C}} \quad ...(19.26)$$

19.7. High and Low Gravity Dams

The principal stress calculated for an elementary profile is given by Equation (19.25), *i.e.* $\sigma = \gamma_w H (S_c - C + 1)$. The value of principal stress calculated above varies only with H, as all other factors are fixed.

To avoid dam failure by crushing, the value of σ should be less than or at the most equal to the maximum allowable compressive stress of dam material. If f represents the allowable stress of the dam material, then the maximum height (H_{max}) which can be obtained in an elementary profile, without exceeding the allowable compressive stresses of the dam material, is given as :

$$f = \gamma_w H (S_c - C + 1)$$

or $$H = \frac{f}{\gamma_w (S_c - C + 1)}$$

The lowest value of H will be obtained when $C = 0$, *i.e.* when uplift is neglected. Hence, for determining the limiting height and to be on a safer side, uplift is neglected.

H_{max} *i.e.* maximum possible height is given as :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)} \quad ...(19.27)$$

Hence, if the height of a dam having an elementary profile of a triangle, is more than that given by the Equation (19.27), the maximum compressive stress generated will exceed the allowable value. In order to keep it safe within limits, extra slopes on the upstream as well as on the downstream, below the limiting height will have to be given, as shown in Fig. 19.15.

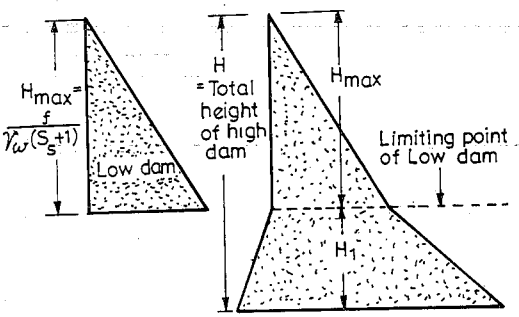


Fig. 19.15

This limiting height (H_{max}) given by Equation (19.27), draws a dividing line between a low gravity dam and a high gravity dram, which are purely technical terms to differentiate between them.

Hence, a low gravity dam is the one whose height is less than that given by Equation (19.27). If the height of the dam is more than this, it is known as a high gravity dam.

The limiting height of a low concrete gravity dam, constructed in concrete having strength equal to 3000 kN/m^2 is thus given :

$$H_{max} = \frac{f}{\gamma_w (S_c + 1)}$$

where $\gamma_w = 9.81\ \text{kN/m}^3$

$S_c = 2.4$

$f = 3000\ \text{kN/m}^2$

$$\therefore \quad H_{max} = \frac{f}{\gamma_w (S_c + 1)} = \frac{3000}{9.81\,(2.4 + 1)} = \mathbf{90\ m}$$

19.8. Profile of a Dam from Practical Considerations

The elementary profile of a gravity dam, (*i.e.* a triangle with maximum water surface at apex) is only a theoretical profile. Certain changes will have to be made in this profile in order to cater to the practical needs. These needs are : (*i*) providing a straight top width, for a road construction over the top of the dam ; (*ii*) providing a free-board above the top water surface, so that water may not spill over the top of the dam due to wave action, etc.

The additions of these two provisions, will cause the resultant force to shift towards the heel. The resultant force, when the reservoir is empty, was earlier passing through the inner middle third point. This will, therefore, shift more towards the heel, crossing the inner middle third point and consequently, tension will be developed at the toe. In order to avoid the development of this tension, some masonry or concrete will have to be added to the upstream side, as shown in Fig. 19.16, which shows the typical section along with the possible dimensions that can be adopted for a low gravity dam section. It should, however, be checked for stability analysis.

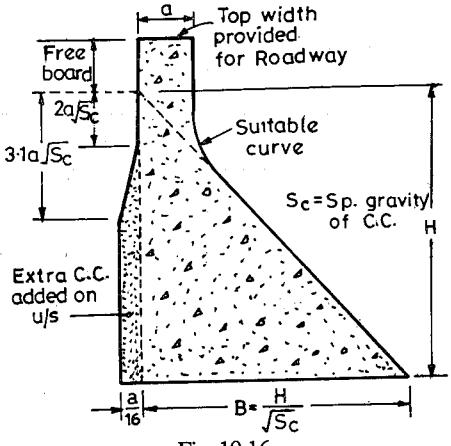


Fig. 19.16.
Typical section of a low gravity dam.

19.9. Design Considerations and Fixing the Section of a Dam

The free-board and top width for roadway should be selected as follows :

(*i*) **Freeboard.** The margin between the maximum reservoir level and top of the dam is known as freeboard. This must be provided in order to avoid the possibility of water spilling over the dam top due to wave action. This can also help as a safety for unforeseen floods, higher than the designed flood.

The freeboard is generally provided equal to $\frac{3}{2}h_w$. where h_w is given by Equations (19.11) and (19.12). However, these days, a *free-board equal to 4 to 5% of the dam height is provided.*

(*ii*) **Top width.** The effects produced by the addition of top width at the apex of an elementary dam profile, and their remedies are explained below :

In Fig. 19.17, let *AEF* be the triangular profile of dam of height H_1. Let the element *ABQA* be added at the apex for providing a top width *a* for a road construction. Let M_1 and M_2 be the inner third and outer third points on base. Thus, AM_1 and AM_2 are the inner third and outer third lines. The weight of the element (W_1) will act through the *e.g.* of this triangle, *i.e.* along *C.M.* Let *CM* and AM_1 cross at *H*, and *CM* and AM_2 cross at *K*.

Reservoir Empty Case. We know that in the elementary profile, the resultant of the forces passes through the inner third point when the reservoir is empty. In such a case, the addition of W_1 above the plane *GHD* will shift this initial resultant towards the downstream side because W_1 lies downstream to the earlier resultant. Similarly, the addition of ΔW_1 'below the plane *GHD* will shift this initial resultant towards the upstream side, causing tension on the downstream. Hence, an upstream batter *GF'* will have to be added below the plane *GHD* as shown in Fig. 19.17.

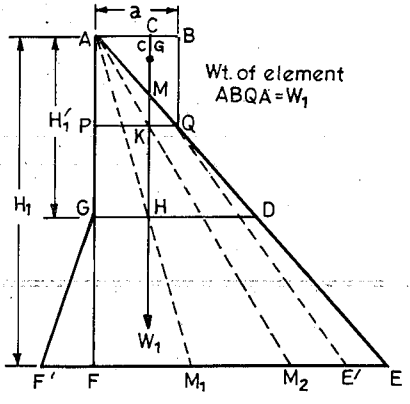


Fig. 19.17

The height H_1' below which this upstream batter is required can be worked out as below :

The ΔAGD, is governed by Equation (19.22), *i.e.*

$$B = \frac{H}{\sqrt{S_c - C}}$$

$$\therefore \quad B_1' = \frac{H_1'}{\sqrt{S_c - C}}$$

But $$GH = \frac{B_1'}{3}$$

$$\therefore \quad GH = \frac{B_1'}{3} = \frac{H_1'}{3\sqrt{S_c - C}}$$

Also $$GH = AC = \frac{2}{3}a$$

$$\therefore \quad \frac{2a}{3} = \frac{H_1'}{3 \cdot \sqrt{S_c - C}}$$

or $$H_1' = 2a \cdot \sqrt{S_c - C} \qquad ...(19.28)$$

Thus, for height greater than H_1', u/s batter is necessary.

Reservoir full case. When the reservoir is full, the resultant of all the forces acting on the elementary profile passes through the outer third point. When W_1 is added to this initial resultant at any plane below the plane *PKQ*, final resultant will shift towards the upstream side because W_1 lies upstream of the initial resultant. In order to bring the resultant back to the outer third point from economy point of view, the slope of the d/s face may be flattened from *QE* to *QE'*.

Thus, an increases in top width, will increase the masonry in the added element and increase it on u/s face, but shall reduce it on d/s face. The most economical top width, without considering earthquake forces has been found by Creager to be equal to 14% of the dam height. Its useful value varies between 6 to 10 m and is generally taken approximately equal to $\sqrt{H}$, where H is the height of max. water level above the bed.

19.10. Design of Gravity Dams

The section of gravity dam should be chosen in such a way that it is the most economical section and satisfies all the conditions and requirements of stability. Hence, after the section of the dam has been arrived at, the stability analysis for the dam must be carried out.

To Decide whether the Dam is Low or High. First of all, the height of the dam to be constructed, should be checked so as to ensure whether it is a low gravity dam or a high gravity dam. If the height of the dam is less than that given by

$$\frac{f}{\gamma_w (S_c + 1)}$$

(where f is the permissible compressive stress of the dam material and S_c is the Sp. gravity of the dam material) ;

then the dam will be a low gravity dam, otherwise it will be a high gravity dam.

Even for a high gravity dam, the upper height $H_1 = \frac{f}{\gamma_w (S_c + 1)}$ can be designed as a low gravity dam and the remaining lower portion can be designed as explained a little later.

Design of Low Dam. The economical section of low gravity dam of height H_1 after deciding the top width a and freeboard, can be drawn as shown in Fig. 19.18.

The base width B_1 of the ΔAIJ can be chosen as given by Equation (19.22) as :

$$B_1 = \frac{H_1}{\sqrt{S_c - C}}$$

or by Equation (19.23), as : $B_1 = \frac{H_1}{\mu (S_c - C)}$

The upstream face can be kept vertical up to a height H_1' to be determined by trial, and whose approximate 1st value may be chosen by Equation (19.28), as :

$$H_1' = 2a \cdot \sqrt{S_c - C}.$$

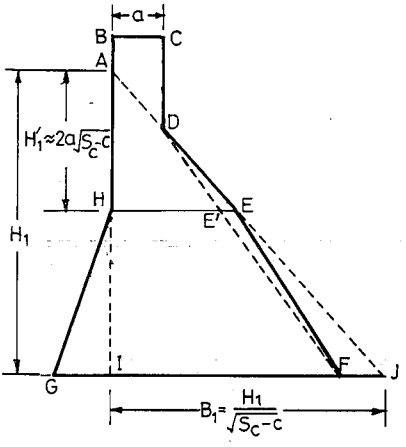


Fig. 19.18

Below this height H_1', the upstream face as well as the downstream face are sloped in such a manner that no tension is developed any-where in the dam, and the resultant forces remain as close to the outer third and inner third points as possible, for reservoir full and reservoir empty cases, respectively. This is accomplished by hit and trial method, and when it is so accomplished, all the stability requirements will be satisfied. The final shape of the dam will then be *ABCDEFGHA*.

The d/s face can be brought inward from the point D itself rather than from the point E (explained earlier). In such a case, the final dam shape will be *ABCDE' FGHA*. Sometimes, in practical conservative designs, the d/s slope is not at all brought inward and is kept the same as is obtained in an elementary profile. The final dam shape, in such a case, will be *ABCDEJGHA*.

Note. However, in most of the practical conservative designs, the low gravity dam is designed as per the provision of Fig. 19.16 and the d/s slope is not brought inward.

Design of High Dam. When the height of the dam exceeds H_1 given by $\frac{f}{\gamma_w (S_c - 1)}$; then its upper height equal to H_1 can be designed as low gravity dam as explained earlier, and its remaining height can be designed by dividing it into a number of suitable strips as shown in Fig. 19.19. The design of each strip can be carried out as per the formulas given below. The basis of these formula is that, the maximum normal stress (*i.e.* principal stress) should not exceed the allowable value (f), and at the same time, the section should be as economical as possible.

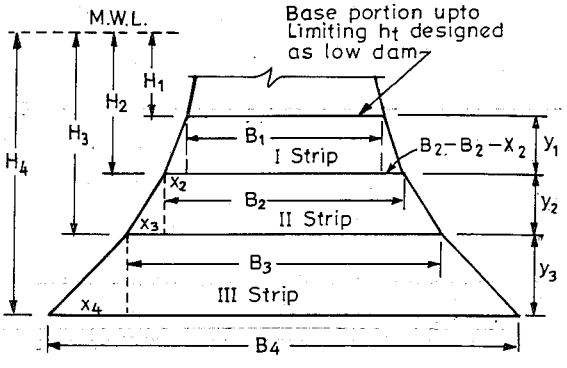


Fig. 19.19

Design of I Strip

The total base width required at the bottom of the 1st strip (B_2) is given by

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_2^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_2^4}{4 \cdot W_2^2}\right]} \quad \text{...[19.29 (a)]}$$

where B_1 = Base width of low dame, *i.e.* the base width at top of 1st strip

B_2 = Base width required at the bottom of 1st strip

H_2 = Height of dam portion from M.W.L. to the bottom of I strip

γ_w = Unit weight of water

f = Allowable compressive stress of the dam material

W_1 = Total vertical weight of dam and water, above the top of I strip

W_2 = Total wt. of dam portion and water above the bottom of I strip.

The increase of base width required on the upstream side, at the bottom of I strip (say X_2), is given by the equation

$$\frac{\gamma_w \cdot S_c \cdot y_1}{24}\left[3 \cdot B_1^2 - B_2^2 + 6 \cdot X_2 (B_1 + B_2) + 2B_1 B_2\right]$$

$$-\frac{\gamma_w \cdot X_2}{12}\left[H_1 + H_2\right]\left[2 \cdot B_2 - 3X_2\right] - W_1\left[\frac{B_2 - B_1}{3} - X_2\right] = 0 \quad \text{...[19.30 (a)]}$$

The increase in width required on u/s, *i.e.* X_2 can be determined from this equation. Total width B_2 is already known. Hence, the increase required on d/s can be calculated as $(B_2 - B_1 - X_2)$.

Design of II Strip

Equation [19.29 (*a*)] for the I strip can be changed to Equation [19.29 (*b*)] for the II strip by changing suffixes as below

$$B_3 = \sqrt{\frac{\gamma_w \cdot H_3^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_3^4}{4W_3^2}\right]} \qquad \text{...[19.29 (b)]}$$

where H_3 = Height of the dam portion from M.W.L. to the bottom of the II strip

W_3 = Total vertical wt. of the dam and water above the bottom of II strip.

Similarly, Equation [19.30 (*a*)] can be written as

$$\frac{\gamma_w \cdot S_c \cdot y_2}{24}\left[3 \cdot B_2^2 - B_3^2 + 6 \cdot X_3 (B_2 + B_3) + 2 \cdot B_2 B_3\right]$$

$$-\frac{\gamma_w \cdot X_3}{12}\left[H_2 + H_3\right]\left[2 \cdot B_3 - 3X_3\right] - W_2\left[\frac{B_3 - B_2}{3} - X_3\right] = 0 \qquad \text{...[19.30 (b)]}$$

Total width B_3 required is known from Equation [19.29 (*b*)] and the increase required on u/s (X_3) is also known from equation [19.30 (*b*)]. The increase required on d/s can be worked out as equal to $(B_3 - B_2 - X_3)$.

All the remaining strips are designed in this way till the base of the dam is reached.

Example 19.2. *Fig. 19.20 (a) shows the section of a gravity dam built of concrete. Examine the stability of this section at the base.*

The earthquake forces may be taken as equivalent to 0.1 g for horizontal forces and 0.05 g for vertical forces. The uplift may be taken as equal to the hydrostatic pressure at the either ends and is considered to act over 60% of the area of the section.

A tail water depth of 6 m is assumed to be present when the reservoir is full and there is no tail water when the reservoir is empty.

Also indicate the values of various kinds of stresses that are developed at heel and toe. Assume the unit wt. of concrete as 24 kN/m³ ; and unit wt. of water = 10 kN/m³.

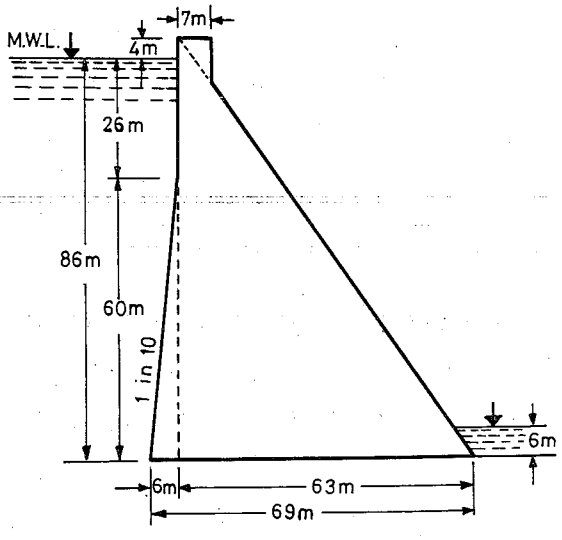


Fig. 19.20 (*a*)

Solution. The stability analysis shall be carried out for both the cases, *i.e.* (1) Reservoir Empty, and (2) Reservoir Full.

Case (I) Reservoir Empty. Consider 1 m length of the dam.

When the reservoir is empty, the various forces are worked out in Table 19.2 (*a*) with reference to Fig. 19.20 (*b*). Horizontal earthquake forces acting towards upstream are considered. Stability is examined for two sub-cases, *i.e.* (*a*) When vertical earthquake

forces are additive to the weight of the dam ; (*b*). When vertical earthquake forces are subtractive to the dam weight.

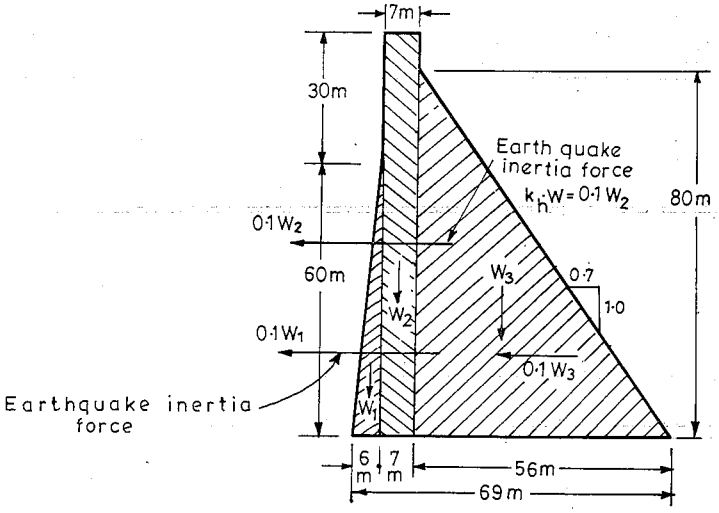


Fig. 19.20 (*b*). Reservoir empty case.

Table 19.2 (*a*)

Name of the force	*Designation if any*	*Magnitude of force in kN.*		*Lever arm m*	*Moments about the toe anti-clockwise (+ve) in kN.m.*
		Vertical	*Horizontal*		
Downward wt. of dam	W_1	(+) $\frac{1}{2} \times 6 \times 60 \times 24 = 4{,}320$		65.0	(+) 2,80,400
	W_2	(+) $7 \times 90 \times 24 = 15{,}110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2} \times 56 \times 80 \times 24 = 53700$		37.33	(+) 20,00,000
		$\Sigma V_1 = 73{,}130$			$\Sigma M_1 =$ (+) 31,79,400
Horizontal earthquake forces	P_{w1}		$0.1\ W_1 = 0.1 \times 4320 = 432$	20.0	(+) 8640
	P_{w2}		$0.1\ W_2 = 0.1 \times 1{,}511 = 1511$	45.0	(+) 68000
	P_{W3}		$0.1\ W_3 = 0.1 \times 5{,}370 = 5370$	26.67	(+) 1,43,200
			$\Sigma H = 7313$		$\Sigma M_2 = 2{,}19{,}840$
Vertical earthquake forces		$\Sigma V_2 = 0.05 \times \Sigma V_1 = 0.05 \times 73130 = 3{,}657$			$\Sigma M_2 = 0.05 \times \Sigma M_1 = 0.05 \times 31{,}79{,}400 = 1{,}58{,}970$

Case (I). (*a*) *Reservoir empty and vertical earthquake forces are acting downward.*
From table 19.2 (*a*), we have $\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3$

$$= 31,79,400 + 2,19,840 + 1,58,970 = \mathbf{35,58,210\ kN \cdot m}$$

Also, $\Sigma V = \Sigma V_1 + \Sigma V_2 = 73,130 + 3,657 = 76,787\ \text{kN}$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{35,58,210}{76,787} = 47.3\ \text{m}$$

$$e = \frac{B}{2} - \bar{x} = \frac{69}{2} - 46.3 = 34.5 - 46.3 = -\mathbf{11.8\ m} > \frac{B}{6},\ \textit{i.e.}\ 11.5\ \text{m}.$$

Resultant acts near the heel and slight tension will develop at toe.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$\therefore \quad p_{max/min} = \frac{76,787}{69}\left[1 \pm \frac{6 \times 11.8}{69}\right] = 1114\ [1 \pm 1.026]$$

p_v at heel $= 1114 \times 2.026 = 2260\ \text{kN/m}^2$; which is ≤ 3000 (safe)

p_v at toe $= 1114 \times (-0.026) = -29\ \text{kN/m}^2$; which is < 420 (*safe*)

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76787}{69} = 1114\ \text{kN/m}^2\ ;\ \text{which is} < 3000\ \text{(safe)}$$

Principal stress at toe,

$$\sigma = p_v \sec^2 \alpha\ ;\ (\tan \alpha = 0.7)$$

$$= -29\ (1 + 0.49) = -29 \times 1.49 = -43\ \text{kN/m}^2\ ;\ \text{which is} < 420\ \text{(safe)}$$

Principle stress at heel

$$\sigma_1 = p_{v \cdot (heel)} \sec^2 \phi \qquad \text{where } \tan \phi = 0.1$$

$$\text{or} \quad \sec^2 \phi = 1 + \tan^2 \phi = 1 + 0.01 = 1.01.$$

or $$\sigma_1 = 2260 \times 1.01 = \mathbf{2280\ kN/m^2}\ ;\ \text{which is} < 3000\ \text{(safe)}.$$

Shear stress at toe

$$\tau_{0(toe)} = p_{v(toe)} \tan \alpha$$

$$= -29 \times 0.7 = -20.3\ \text{kN/m}^2\ ;\ \text{which is} < 420\ \text{(safe)}$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v \cdot (heel)} \tan \phi$$

$$= 2260 \times 0.1 = 226\ \text{kN/m}^2\ ;\ \text{which is} < 3000\ \text{(safe)}.$$

Case I. (*b*) *Reservoir empty and vertical earthquake forces are acting upward.*

Then $\Sigma V = \Sigma V_1 - \Sigma V_3$

$$= 73,130 - 3657 = 69473\ \text{kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 - \Sigma M_3$$

$$= 31,79,400 + 2,19,840 - 1,58,970 = 32,40,270\ \text{kN} \cdot \text{m}.$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{32,40,270}{69473} = \mathbf{46.7\ m}.$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 46.7 = (-)\ 12.2\ \text{m} < \frac{B}{6}$$

[– ve sign shows that resultant lies near the heel and, therefore, tension will develop at toe.]

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{69{,}473}{69} = 1004 \text{ kN/m}^2$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{69473}{69}\left[1 \pm \frac{6 \times 12.2}{69}\right] = 1004\,[1 \pm 1.06]$$

p_v at heel $= 1004 \times 2.06 = 2070 \text{ kN/m}^2 < 3000$ (safe)

p_v at toe $= (-)\ 1004 \times 0.06 = -60.3 \text{ kN/m}^2 < 420$ (safe)

Principal stress at toe

$$= \sigma = p_{v(toe)} \sec^2 \alpha$$

$$= -60.3\,(1 + 0.49) = -60.3 \times 1.49 = 90 \text{ kN/m}^2$$

Shear stress at toe

$$= \tau_0 = p_{v(toe)} \tan \alpha = -60.3 \times 0.7$$

$$= \mathbf{-42.21 \text{ kN/m}^2}\text{ ; which is } < 420 \text{ (safe)}$$

stresses at heel remain critical in this 1st case.

Case II. When the reservoir is full

Horizontal earthquake moving towards the reservoir causing upstream acceleration, and thus producing horizontal forces towards downstream is considered, as it is the worst case for this condition. Similarly, a vertical earthquake moving downward and thus, producing forces upward, *i.e.* subtractive to the weight of the dam is considered.

The uplift coefficient C is taken as equal to 0.6, as given in the equation, and thus uplift pressure diagram as shown in Fig. 19.20 (*c*), is developed.

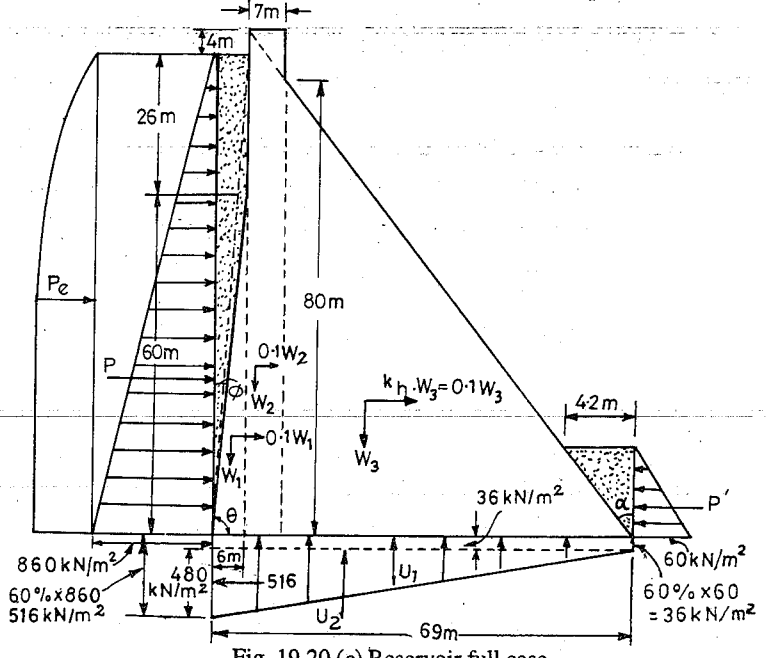


Fig. 19.20 (*c*) Reservoir full case.

The various forces acting in this case are :

(*i*) Hydrostatic pressures P and P'.

(*ii*) Hydrodynamic pressure P_e. (P_e' is neglected as it is very small and neglection is on conservative side.)

(*iii*) Uplift forces U_1 and U_2

(*iv*) Weight of the dam, W_1, W_2 and W_3.

(*v*) Horizontal inertial earthquake forces acting towards downstream, equal to 0.1 W_1, 0.1 W_2 and 0.1 W_3 at *c.g.s.* of these weights W_1, W_2 and W_3 respectively.

(*vi*) A vertical force equal to 0.05 W or (0.05 ΣV_1) acting upward.

Calculation of P_e

P_e and the moment due to this hydrodynamic force is calculated, and then all the forces and their moments are tabulated in Table 19.2 (*b*).

Calculation of P_e from Zanger's formulas

$$P_e = 0.726\, p_e H \qquad ...(19.3)$$

$$\text{where } p_e = C_m \cdot K_h \cdot \gamma_w \cdot H \qquad ...(19.4)$$

$$\text{and} \quad C_m \doteq 0.735 \frac{\theta}{90°}$$

Since the u/s inclined face is extended for more than half the depth, the overall slope up to the whole height may be taken.

$$\therefore \quad \tan\theta = \frac{86}{6} = 14.33$$

$$\theta = 81.9°$$

$$\therefore \quad C_m = 0.735 \times \frac{81.9°}{90°} = 0.668.$$

$$p_e = 0.668 \times 0.1 \times 10 \times 86 = 57.5$$

$$P_e = 0.726 \times 57.5 \times 86 = \mathbf{3580\ kN.}$$

$$M_e = 0.412 \cdot P_e \cdot H = 0.412 \times 3580 \times 86 = \mathbf{1,26,500\ kN.m.}$$

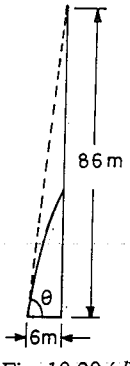


Fig. 19.20 (*d*)

Case 2 (*a*) Reservoir full with all forces including uplift

$$\Sigma M = [31,79,400 + 2,23,380 - 8,47,500 - 1,58,970 - 10,59,730 - 1,26,500 - 2,19,840]$$

$$= \mathbf{34,02,780 - 24,12,540 = 9,90,240\ kN/m.}$$

$$\Sigma V = 73130 + 3486 - 19030 - 3657 = 53929\ \text{kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{9,90,240}{53,929} = 18.36\ \text{m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 18.36 = 16.14 > \frac{B}{6}$$

The resultant is nearer the toe and tension is developed at the heel.

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{53929}{69} = 782\ \text{kN/m}^2.$$

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

Table 19.2 (*b*)

Name of force	*Designation if any*	*Magnitude of force in kN*		*Lever arm in m*	*Moments about toe in kN. Anticlock wise (+ve) and clockwise (– ve) in kN.m*
		Vertical forces Downward = +ve Upward = –ve	*Horizontal forces Towards Upstream = +ve Towards Downstream = –ve*		
(1)	(2)	(3)	(4)	(5)	(6)
Weight of Dam	W_1	(+) $\frac{1}{2}\times 6\times 60\times 1\times 24 = 4320$		65.0	(+) 2,80,400
	W_2	(+) $7\times 90\times 1\times 24 = 15,110$		59.5	(+) 8,99,000
	W_3	(+) $\frac{1}{2}\times 56\times 80\times 1\times 24 = 53,700$		37.33	(+) 20,00,000
		$\Sigma V_1 = (+)\ 73,130$			$\Sigma M_1 = 31,79,400$
Weight of water supported on u/s slope	—	(+) $26\times 6\times 1\times 10 = 1560$		66.0	(+) 1,02,800
	—	(+) $\frac{1}{2}\times 60\times 6\times 1\times 10 = 1800$		67.0	(+) 1,10,400
water on d/s slope.	—	(+) $\frac{1}{2}\times 6\times 4.2\times 1\times 10 = 126$		1.4	(+) 180
		$\Sigma V_2 = (+)\ 3486$			$\Sigma M_2 = (+)\ 2,23,380$
Uplift forces	U_1	(–) $69\times 3.6\times 10 = 2,480$		34.5	(–) 85,500
	U_2	(–) $\frac{1}{2}\times 69\times 48\times 10 = 16,550$		46.0	(–) 7,62,000
		$\Sigma V_3 = (-)\ 19,030$			$\Sigma M_3 = (-)\ 8,47,500$
Upward vertical earthquake forces 0.05 *W*		$\Sigma V_4 = (-)\ 0.05 \cdot \Sigma V_1$ $= (-)\ 0.05\times 73,130$ $= (-)\ 3,657$			$= (-)\ 0.05 \cdot \Sigma M_1$ $= (-)\ 0.05\times 31,79,400$ $\Sigma M_4 = (-)\ 1,58,970$
Horizontal hydrostatic pressure	P		(–) $\frac{1}{2}\times 10\times 86\times 86\times 1$ $= (-)\ 36,980$	28.67	(–) 10,60,090
	P'		(+) $\frac{1}{2}\times 10\times 6\times 6\times 1 = (+)\ 180$	2.0	(–) 360
			$\Sigma H_1 = (-)\ 36,800$		$\Sigma M_5 = (-)\ 10,59,730$
Horizontal hydro-dynamic pressure	P_e		Calculated separately earlier : $= (-)\ 3,580$ $\Sigma H_2 = (-)\ 3,580$		$\Sigma M_e = (-)\ 1,26,500$ (calculated separately earlier
Horizontal inertia forces due to earthquake	P_{W_1}		(–) $0.1\ W_1 = (-)\ 432$	20.0	(–) 8,640
	P_{W_2}		(–) $0.1\ W_2 = (-)\ 1,511$	45.0	(–) 68,000
	P_{W_3}		(–) $0.1\ W_3 = (-)\ 5,370$	26.67	(–) 1,43,200
			$\Sigma H_3 = (-)\ 7,313$		$\Sigma M_7 = (-)\ 2,19,840$

$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3 = (-)36,800 - 3580 - 7313 = (-)\ 47,693$

$$= \frac{53929}{69}\left[1 \pm \frac{6 \times 18.32}{69}\right] = 782\,[1 \pm 1.595]$$

p_v (at toe) $= 782 \times 2.595 = \mathbf{2030\ kN/m^2}$; which is $< 3000\ \mathbf{kN/m^2}$ (∴ Safe)

p_v (at heel) $= -782 \times 0.405$

$= \mathbf{-316.7\ kN/m^2}$; which is $< 420\ kN/m^2$ (∴ Safe)

Since the tensile stress developed is less than the safe allowable value, the dam is safe even when examined with seismic forces, under reservoir full condition.

Principal stress at toe

$= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha$ *i.e.* Eq. (19.17)

where $\tan \alpha = 0.7$, $p' = 60\ kN/m^2$; $p_v = 2030\ kN/m^2$

$\sigma = 2030\,(1 + \tan^2 \alpha) - p' \tan^2 \alpha$

$= 2030\,(1 + 0.49) - 60 \times 0.49 = 2030 \times 1.49 - 29$

$= 3025 - 29 = \mathbf{2996\ kN/m^2}$ **: which is < 3000** (just Safe)

Principal stress at heel is

$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi$ *i.e.* Eq. (19.19)

where ϕ is the angle which the upstream face makes with the vertical

$\tan \phi = 0.1$

∴ $\sigma_1 = -316.7\,[1 + (0.1)^2] - (860 + 57.5)\,(0.1)^2$

$= -316.7 \times 1.01 - 917.5 \times 0.01 = -319.9 - 9.2$

$= \mathbf{-329.1\ kN/m^2}$; which is $< 420\ kN/m^2$ (Hence, safe)

Shear stress at toe

$\tau_{0(toe)} = (p_{v(toe)} - p') \tan \alpha = (2030 - 60)\ 0.7$

$= 1970 \times 0.7 = \mathbf{1379\ kN/m^2}$.

Shear Stress at heel

$\tau_{0(heel)} = -[p_{v(heel)} - (p + p_e)] \tan \phi$

$= -[-329.1 - (860 + 57.5)]\ 0.1$

$= -[-329.1 - 917.5]\ 0.1 = +1246.6 \times 0.1 = \mathbf{124.7\ kN/m^2}$

Factor of safety against overturning

$= \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = \frac{34,02,780}{24,12,540} = 1.41$; which is < 1.5 (Hence, Unsafe)

Factor of safety against sliding

$= \frac{\mu \cdot \Sigma V}{\Sigma H}$

where $\mu = 0.7$

$\Sigma V = 53,929$

$\Sigma H = \Sigma H_1 + \Sigma H_2 + \Sigma H_3$

$= -36800 - 3580 - 7313 = -47,693\ kN$

$$\textit{Sliding factor} = \frac{0.7 \times 53929}{47693} = 0.79, \text{ which is} < 1 \qquad \text{(Hence, Unsafe)}$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \cdot \Sigma V + B \cdot q}{\Sigma H}$$

$$= \frac{0.7 \times 53929 + 69 \times 1400}{47693}$$

$$= 2.81 \text{ ; which is less than 3} \qquad \text{(Hence, slightly unsafe)}$$

Case 2 (*b*). Reservoir full, without uplift

Sometimes, values of stresses at toe and heel are worked out when there is no uplift, as the vertical downward forces are maximum in this case. For this case, we shall calculate ΣM and ΣV by ignoring the corresponding values of ΣV_3 and ΣM_3 caused by uplift.

$$\therefore \quad \Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_4 + \Sigma M_5 + \Sigma M_6 + \Sigma M_7$$

$$= 31,79,400 + 2,23,380 - 1,58,970 - 10,59,730 - 1,26,500 - 2,19,840$$

$$= 34,02,780 - 15,65,040 = 18,37,740$$

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_4 = 73130 + 3486 - 3657 = 72,959 \text{ kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{18,37,740}{72,959} = 25.19 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 25.9 = 9.31 \text{ m} > \frac{B}{6} \quad i.e. \quad \frac{69}{6} = 11.5 \text{ m}$$

Resultant is nearer the toe and no tension is developed any where.

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{72,959}{69}\left[1 \pm \frac{6 \times 9.31}{69}\right] = 1057\,[1 \pm 0.81]$$

p_v at toe $= 1057 \times 1.81 = \mathbf{1913\ kN/m^2} < 3000$ (∴ Safe)

p_v at heel $= 1057 \times 0.19 = \mathbf{201\ kN/m^2} < 3000$ (∴ Safe)

Principal stress at toe $= \sigma = p_v \cdot \sec^2 \alpha - p' \tan^2 \alpha$ ...(19.17)

$p' = 60$, $\tan \alpha = 0.7$

$$\therefore \quad \sigma = 1913\,(1 + 0.49) - 60 \times 0.49 = 1913 \times 1.49 - 29 = 2821 \text{ kN/m}^2 < 3000$$

(Hence, Unsafe)

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - (p + p_e) \tan^2 \phi \qquad \text{...(19.19)}$$

where $\tan \phi = 0.1$

$$\therefore \quad \sigma_1 = 201(1 + 0.01) - (860 + 57.5) \times 0.01$$

$$= 203 - 9 = 194 \text{ kN/m}^2 < 420 \quad \text{(Safe)}$$

Shear stress at toe

$$\tau_0 = (p_v - p') \tan \alpha \qquad i.e. \text{ Eq. (19.20)}$$

$$= (1913 - 60)\,0.7$$

$$= 1853 \times 0.7 = 1297 \text{ kN/m}^2 < 1400 \qquad (\therefore \text{ safe})$$

Note. Shear friction factor, etc. are not worked out here as they were more critical in the 1st case, *i.e.* in 'Reservoir full with uplift' case.

Conclusion. The dam is unsafe only in sliding and S.F.F., for which shear key etc. can be provided.

Example 19.3. *Examine the stability of the dam section given in the previous example, if there are no seismic forces acting on the dam. Also state the magnitude of maximum compressive stress and maximum shear stress that may develop under any conditions of loading in the dam and also state whether tension is developed anywhere or not.*

Solution. The figures calculated earlier in Table 19.2 (*a*) and (*b*) shall be used here.

Case I. When the reservoir is empty

$$\Sigma V = \Sigma V_1 \text{ from Table 10.2 } (a) = 73130$$

$$\Sigma M = \Sigma M_1 \text{ from Table 19.2 } (b) = 3179400$$

$$\therefore \quad \bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{3179400}{73130} = 43.4 \text{ m}$$

$$e = \frac{B}{2} - \bar{x} = 34.5 - 43.4 = -8.9 \text{ m}$$

–ve sign means that the resultant is towards left side, *i.e.* nearer to the heel, and since $e < \frac{B}{6}$, no tension is developed

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$

$$= \frac{73130}{69}\left[1 \pm \frac{6 \times 8.9}{69}\right]$$

$$= 1060\,[1 \pm 0.774]$$

$$p_v \text{ at heel} = 1060(1 + 0.774) = 1060 \times 1.774 = \mathbf{1880\ kN/m^2}$$

$$p_v \text{ at toe} = 1060(1 - 0.774) = 1060 \times 0.226 = \mathbf{239\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{73130}{69} = \mathbf{1060\ kN/m^2}$$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha$$

$$= 239(1 + 0.49) = 239 \times 1.49 = \mathbf{357\ kN/m^2}$$

Principal stress at heel,

$$\sigma = p_{v(heel)} \sec^2 \phi$$

where $\tan \phi = 0.1$

$$= 1880(1 + 0.01) = 1880 \times 1.01 = \mathbf{1896\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = p_{v(toe)} \tan \alpha$$

$$= 239 \times 0.7 = 167.3 \text{ kN/m}^2$$

Shear stress at heel

$$\tau_{0(heel)} = p_{v(i.e.,\,l)} \tan \phi$$
$$= 1880 \times 0.1 = 188 \text{ kN/m}^2.$$

Case II (*a*). **When the reservoir is full with full uplift**

The values of forces, and moments etc. shall be used from the already calculated values in Table 19.2 (*b*).

$$\Sigma V = \Sigma V_1 + \Sigma V_2 + \Sigma V_3$$

(Seismic forces absent mean ΣV_4 and ΣM_4 are zero)

$$= 73130 + 3486 - 19030 = 76616 - 19030 = 57586 \text{ kN}$$

$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_3 + \Sigma M_5$$
$$= 31,79,400 + 2,23,380 - 8,47,500 - 10,59,730$$
$$= 34,02,780 - 19,07,230 = 14,95,550 \text{ kN–m.}$$

$$\Sigma H = \Sigma H_1 = -36800 \text{ kN}$$

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{14,95,550}{57,586} = \mathbf{26 \text{ m}}$$

$$\frac{B}{2} - \bar{x} = 34.5 - 26 = \mathbf{8.5 \text{ m}}$$

The resultant is nearer the toe and e is less than $\frac{B}{6}$, and hence, no tension is developed anywhere

$$\therefore \quad p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{57586}{69}\left[1 \pm \frac{6 \times 8.5}{69}\right] = 835[1 \pm 0.738]$$

p_v at toe $= 835(1 + 0.738) = 835 \times 1.738 = \mathbf{1451 \text{ kN/m}^2}$

p_v at heel $= 835(1 - 0.738) = 835 \times 0.262 = \mathbf{219 \text{ kN/m}^2}$

Principal stress at toe

$$\sigma = p_{v(toe)} \sec^2 \alpha - p' \tan^2 \alpha \qquad \text{...(19.17)}$$
$$= 1451(1 + 0.49) - 60 \times 0.49 = 1451 \times 1.49 - 29$$
$$= 2162 - 29 = \mathbf{2133 \text{ kN/m}^2.}$$

Principal stress at heel

$$\sigma_1 = p_{v(heel)} \sec^2 \phi - p \tan^2 \phi$$

where $\sec \phi = 0.1$

$$\sigma_1 = 219 \times (1 + 0.01) - (860) \times 0.01 = 221.2 - 8.6 = \mathbf{212.6 \text{ kN/m}^2.}$$

Note. This is the minor principal stress and major principal stress is equal to p, *i.e.* 860 kN/m^2.

Shear stress at toe

$$\tau_0 = \left[p_{v(toe)} - p'\right] \tan \alpha = (1451 - 60)\, 0.7 = 974 \text{ kN/m}^2.$$

Shear stress at heel

$$\tau_{0(heel)} = -(p_v - p)\tan\phi$$
$$= -(219 - 860)0.1 = 541 \times .0.1 = 54.1 \text{ kN/m}^2;$$

Factor of safety against overturning

$$= \frac{\Sigma M\,(+)}{\Sigma M\,(-)} = \frac{3402780}{1907230} = 1.78 > 1.5 \qquad (\therefore \text{ Safe})$$

Factor of safety against sliding

$$= \frac{\mu \Sigma V}{\Sigma H} = 0.7 \times \frac{57586}{36800} = 1.10 > 1 \qquad (\therefore \text{ Safe})$$

Shear friction factor

$$\text{S.F.F.} = \frac{\mu \Sigma V + B \cdot q}{\Sigma H} = \frac{0.7 \times 57586 + 69 \times 1400}{36800}$$
$$= 3.72 < 4 \text{ to } 5 \qquad (\therefore \text{ Unsafe})$$

Case II (*b*). When the reservoir is full without uplift

$$\Sigma V = \Sigma V_1 + \Sigma V_2 = 73130 + 3486 = 76616 \text{ kN}$$
$$\Sigma M = \Sigma M_1 + \Sigma M_2 + \Sigma M_5 = 3179400 + 223380 - 1059730$$
$$= 3402780 - 1059730 = 2343050 \text{ kN–m}$$
$$\Sigma H = \Sigma H_1 = (-)\,36800 \text{ kN}$$
$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{2343050}{76616} = 30.6 \text{ m}$$
$$e = \frac{B}{2} - \bar{x} = 34.5 - 30.6 = \mathbf{3.9\ m}$$

Resultant is nearer the toe and $e < \frac{B}{6}$; hence, no tension is developed anywhere

$$p_{max/min} = \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right]$$
$$= \frac{76616}{69}\left[1 \pm \frac{6 \times 3.9}{69}\right]$$
$$= 1110\ [1 \pm 0.339]$$
$$p_v \text{ at toe} = 1110 \times 1.339 = \mathbf{1490\ kN/m^2}$$
$$p_v \text{ at heel} = 1110 \times 0.661 = \mathbf{734\ kN/m^2}$$

Average vertical stress

$$= \frac{\Sigma V}{B} = \frac{76619}{69} = \mathbf{1110\ kN/m^2}$$

Principal stress at toe,

$$\sigma = p_{v(toe)} \sec^2\alpha - p' \tan^2\alpha$$
$$= 1490\,(1 + 0.49) - 60 \times 0.49$$
$$= 2220 - 29 = \mathbf{2191\ kN/m^2}$$

Principal stress at heel,

$$\sigma_1 = p_{v(heel)} \sec^2\phi - p \tan^2\phi$$

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\ 0.7 = 1430 \times 0.7 = 1001\ kN/m^2$$

Shear stress at heel

$$= -[p_{v(heel)} - p] \tan \phi$$
$$= -[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam	$= 1001\ kN/m^2$.	**Ans.**
Maximum compressive stress developed in dam	$= 2191\ kN/m^2$	
No tension is developed anywhere.		
Factor of safety against sliding	= 1.10	
S.F.F.	= 3.72	
Factor of safety against overturning	= 1.78	

Example 19.4. *Design a concrete gravity dam for the following data :*

Maximum allowable compressive stress in concrete	$= 3000\ kN/m^2$.
Maximum reservoir level	*= 200.0 m.*
R.L. of bottom of dam	*= 100.0 m*
Specific gravity of concrete	*= 2.4.*
Unit wt. of water	$= 10\ kN/m^3$.

Solution. The practical profile of dam will have a freeboard of about say 3 to 4% of dam height. Use 3 m as freeboard. The freeboard cannot be calculated as the wave height, etc. are not given.

The R.L. of the top of the dam

$$= 200.0 + 3.0 = 203.0\ m.$$

The height of the low gravity dam

$$= H_1 = \frac{f}{\gamma_w\,(S_c + 1)} \qquad ...(19.26)$$

In this question,

$$f = 3000\ kN/m^2\ (i.e.\ 30\ kg/cm^2)$$
$$\gamma_w = 10\ kN/m^3$$
$$S_c = 2.4$$

$$\therefore \quad H_1 = \frac{3000}{10\,(2.4 + 1)} = 88.2\ m,\ < \text{Height of dam.}$$

Therefore, it is a high gravity dam.

Hence, the dam from RL 200.0 m to RL 200 – 88.2 = 111.8 m shall be designed as a low gravity dam, and the remaining bottom height of the dam from RL 111.8 m to 100.0 m shall be designed on the principles of high gravity dam.

Design of low dam between RL 200.0 m to RL 111.8

A section for this dam can be chosen as per the provisions of Fig. 19.16.

$$\text{Top width required} = a = \sqrt{\frac{H_1}{3.28}} = \sqrt{\frac{88.2}{3.28}} = \sqrt{26.9} = 5.18 \text{ m}$$

$$\text{Base width required} = \frac{H}{\sqrt{S}} = \frac{88.2}{\sqrt{2.4}} = 56.8 \text{ m}.$$

The upstream projection from the vertical face required

$$= \frac{a}{16} = \frac{5.18}{16} = 0.33 \text{ m (say)}$$

Total base width (B_1) provided

$$= 56.8 + 0.33 = 57.13 \text{ m}.$$

The u/s. batter starts at a depth of

$$2a\sqrt{S_c} = 2 \times 5.18 \times \sqrt{2.4} = 16.1 \text{ m}.$$

from below the M.W.L. and it ends at a depth of

$$3.1\, a\sqrt{S_c} = 3.1 \times 5.18\sqrt{2.4} = 24.9 \text{ m}$$

below the M.W.L. The section of this portion of dam is shown in Fig. 19.21 (*a*).

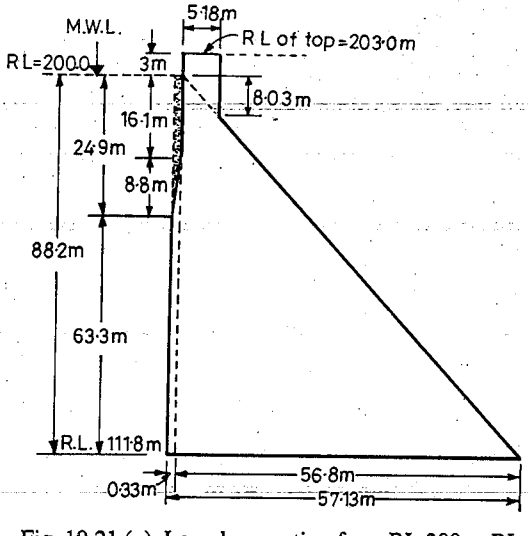


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

Design of the Dam from RL 111.8 m to RL 100 m

Let us divide this length of 11.8 m into 3 strips. Let the depth of the I strip be 3.8 m and the depths of II and III strips be 4.0 m each.

Design of the 1st strip

Consider 1 m length of the dam.

$$= 734\,(1 + 0.01) - 860 \times 0.01$$
$$= 742 - 9 = \mathbf{733\ kN/m^2}$$

Shear stress at toe

$$\tau_0 = [p_{v(toe)} - p'] \tan \alpha$$
$$= (1490 - 60)\ 0.7 = 1430 \times 0.7 = 1001\ \text{kN/m}^2$$

Shear stress at heel

$$= -[p_{v(heel)} - p] \tan \phi$$
$$= -[734 - 860] \times 0.7 = 126 \times 0.7 = \mathbf{88.2\ kN/m^2}$$

Conclusions. We find that the dam is safe throughout except that the S.F.F. is equal to 3.72, while generally it should be between 4 to 5. The dam thus remains slightly unsafe in S.F.F. even when the seismic forces are not considered.

The results of stability analysis are given below :

The maximum shear stress developed in dam $= 1001\ \text{kN/m}^2$.
Maximum compressive stress developed in dam $= 2191\ \text{kN/m}^2$
No tension is developed anywhere.
Factor of safety against sliding $= 1.10$
S.F.F. $= 3.72$
Factor of safety against overturning $= 1.78$

Ans.

Example 19.4. *Design a concrete gravity dam for the following data :*

Maximum allowable compressive stress in concrete $= 3000\ kN/m^2$.
Maximum reservoir level $= 200.0\ m$.
R.L. of bottom of dam $= 100.0\ m$
Specific gravity of concrete $= 2.4$.
Unit wt. of water $= 10\ kN/m^3$.

Solution. The practical profile of dam will have a freeboard of about say 3 to 4% of dam height. Use 3 m as freeboard. The freeboard cannot be calculated as the wave height, etc. are not given.

The R.L. of the top of the dam

$$= 200.0 + 3.0 = 203.0\ \text{m}.$$

The height of the low gravity dam

$$= H_1 = \frac{f}{\gamma_w\,(S_c + 1)} \qquad \text{...(19.26)}$$

In this question,

$$f = 3000\ \text{kN/m}^2\ (i.e.\ 30\ \text{kg/cm}^2)$$
$$\gamma_w = 10\ \text{kN/m}^3$$
$$S_c = 2.4$$

$$\therefore \quad H_1 = \frac{3000}{10\,(2.4 + 1)} = 88.2\ \text{m},\ < \text{Height of dam.}$$

Therefore, it is a high gravity dam.

Hence, the dam from RL 200.0 m to RL 200 – 88.2 = 111.8 m shall be designed as a low gravity dam, and the remaining bottom height of the dam from RL 111.8 m to 100.0 m shall be designed on the principles of high gravity dam.

Design of low dam between RL 200.0 m to RL 111.8

A section for this dam can be chosen as per the provisions of Fig. 19.16.

$$\text{Top width required} = a = \sqrt{\frac{H_1}{3.28}} = \sqrt{\frac{88.2}{3.28}} = \sqrt{26.9} = 5.18 \text{ m}$$

$$\text{Base width required} = \frac{H}{\sqrt{S}} = \frac{88.2}{\sqrt{2.4}} = 56.8 \text{ m}.$$

The upstream projection from the vertical face required

$$= \frac{a}{16} = \frac{5.18}{16} = 0.33 \text{ m (say)}$$

Total base width (B_1) provided

$$= 56.8 + 0.33 = 57.13 \text{ m}.$$

The u/s. batter starts at a depth of

$$2a\sqrt{S_c} = 2 \times 5.18 \times \sqrt{2.4} = 16.1 \text{ m}.$$

from below the M.W.L. and it ends at a depth of

$$3.1\, a\sqrt{S_c} = 3.1 \times 5.18\sqrt{2.4} = 24.9 \text{ m}$$

below the M.W.L. The section of this portion of dam is shown in Fig. 19.21 (*a*).

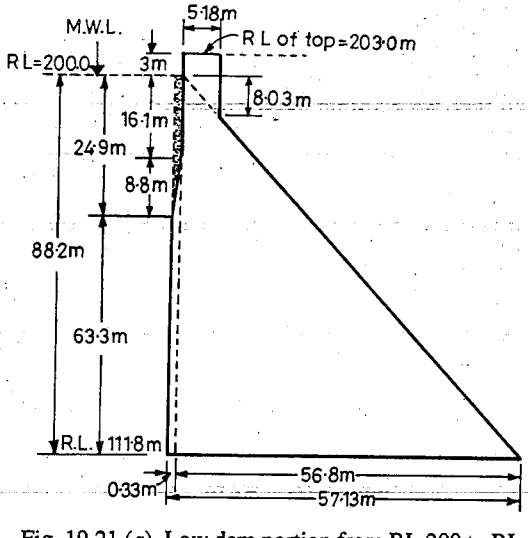


Fig. 19.21 (*a*). Low dam portion from RL 200 to RL 111.8.

Design of the Dam from RL 111.8 m to RL 100 m

Let us divide this length of 11.8 m into 3 strips. Let the depth of the I strip be 3.8 m and the depths of II and III strips be 4.0 m each.

Design of the 1st strip

Consider 1 m length of the dam.

Weight of dam section up to RL 111.8 m. (i.e. weight of low dam)

$$= 24 \times 1 \left[\frac{1}{2} \times 8.8 \times 0.33 + 63.3 \times 0.33 + \frac{1}{2} + 56.8 \times 88.2 + 3 \times 5.18 + \frac{1}{2} \times 8.03 \times 5.18 \right]$$

$$= 24\,[1.5 + 20.9 + 2{,}505 + 15.5 + 20.8]$$

$$= 24\,[2{,}563.7] = \mathbf{61{,}600\ kN.}$$

Approximate width of bottom of 1st strip (say B_2') is obtained by drawing a horizontal line at RL 108 m and by producing the already provided d/s face and u/s face of low dam, as shown in Fig. 19.21 (*b*).

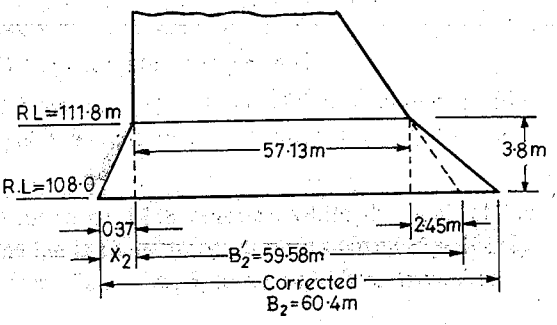


Fig. 19.21 (*b*)

$$B_2' = 57.13 + \frac{56.8}{88.2} \times 3.8$$

$$= 57.13 + 2.45 = 59.58 \text{ m}$$

Approximate weight of 1st strip

$$= 24 \times 1 \left[\frac{57.13 + 59.58}{2} \right] \times 3.8$$

$$= 24 \times 58.35 \times 3.8 = 5320 \text{ kN}$$

Weight of water resting on the u/s face

$$= 10 \times 1 \left[16.1 \times 0.33 + \frac{1}{2} \times 8.8 \times 0.33 \right]$$

$$= 10 \times 0.33 \times 20.5 = 67.7 \text{ ; } \mathbf{say\ 68\ kN.}$$

Total weight of dam and water at top of 1st strip, *i.e.* at base of small dam

$$= W_1 = 61{,}600 + 68 = 61{,}668 \text{ kN ; } \mathbf{Say\ 61670\ kN}$$

Total approximate weight of dam and water at base of 1st strip

$$= W_2 = 61{,}670 + \text{App. weight of 1st strip}$$

$$= 61{,}670 + 5320 = \mathbf{66{,}990\ kN.}$$

The correct base width B_2, which shall keep the maximum compressive stresses within the allowable limits is given by equation [19.29 (*a*)], *i.e.*

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_2^3}{f} \left[1 + \frac{\gamma_w^2 \cdot H_2^4}{4 \cdot W_2^2} \right]}$$

where $H_2 = 92$ m

$W_2 = 66{,}990$ kN

$f = 3000$ kN/m^2

$\gamma_w = 10$ kN/m^3.

Substituting, we get

$$B_2 = \sqrt{\frac{10 \times (92)^3}{3000} \left[1 + \frac{10^2 \cdot (92)^4}{4 \times (66{,}990)^2} \right]}$$

$$= \sqrt{2596\,(1.4)} = \sqrt{3{,}634} = \mathbf{60.4\ m}$$

Now let us find out the projection X_2 of base B_1 on the u/s side. It is given by equation [19.30 (*a*)], *i.e.*

$$\frac{\gamma_w \cdot S_s \cdot y_1}{24}\left[3B_1^2 - B_2^2 + 6X_2(B_1 + B_2) + 2B_1B_2\right]$$
$$-\frac{\gamma_w \cdot X_2}{12}[H_1 + H_2][2B_2 - 3X_2] - W_1\left[\frac{B_2 - B_1}{3} - X_2\right] = 0.$$

where $y_1 = 3.8$ m
$B_1 = 57.13$ m
$B_2 = 60.4$ m
$H_1 = 88.2$ m
$H_2 = 92$ m
$W_1 = 61{,}670$ kN

Substituting, we get

$$\frac{10 \times 2.4 \times 3.8}{24}[3(57.13)^2 - (60.4)^2 + 6X_2(57.13 + 60.4) + 2 \times 57.13 \times 60.4]$$
$$-\frac{10 \cdot X_2}{12}[88.2 + 92][2 \times 60.4 - 3X_2] - 61{,}670\left[\frac{60.4 - 57.13}{3} - X_2\right] = 0.$$

or $3.8[9{,}780 - 3{,}640 + 705X_2 + 6{,}900] - 10X_2[1{,}815 - 45X_2] - 61{,}670[1.09 - X_2]$

or $4.960 + 268X_2 - 1{,}815X_2 + 45X_2^2 - 6{,}720 + 6{,}167X_2 = 0$

or $45X_2^2 + 4{,}620X_2 - 1{,}760 = 0$

Solving this equation, we get

$X_2 = 0.37$ m (say).

Hence, out of 60.4 m base width at base of 1st strip, 0.37 m shall be provided on u/s side as shown in Fig. 19.21 (*b*). This will ensure that the resultant shall pass through the inner third point when the reservoir is empty and no tension will develop at toe.

Design of II Strip. It shall be designed on the same lines on which 1st strip has been designed.

Corrected weight of 1st strip

$$= 24 \times 1\left[\frac{57.13 + 60.4}{2}\right] \cdot 3.8 = 1.2 \times 117.53 \times 3.8 = 5370 \text{ kN}.$$

Weight of water resting on the u/s face of first strip

$$= 10 \cdot \left[88.2 \times 0.37 + \frac{1}{2} \times 3.8 \times 0.37\right]$$
$$= 3.7[88.2 + 1.9] = 383 \text{ kN}$$

Total corrected weight at the base of 1st strip

$$W_2 = 61{,}670 + 5370 + 383$$
$$= 67{,}423 \text{ kN. } \textbf{Say 67,420 kN.}$$

Depth of II strip $= y_2 = 4.0$ m.

Approximate base width B_3' of the bottom of II strip

$$= 60.4 + \frac{56.8}{88.2} \times 4.0 + \frac{0.37}{3.8} \times 4.0$$

$$= 60.4 + 2.58 + 0.39 = 63.37\text{ ; say } \mathbf{63.4\ m.}$$

Approximate weight of II strip

$$= 24 \times 1 \left[\frac{60.4 + 63.4}{2}\right] \times 4 = 5940\text{ kN}$$

Approximate weight of water resting on the u/s face of II strip, if the u/s slope given to 1st strip is extended in the II strip

$$= \left[\frac{92 + 96}{2} \times 0.39\right] 10 = \mathbf{370\ kN}$$

Hence, the total approximate weight of dam and water at base of II strip

$$= W_3 = 67{,}420 + .5940 + 370 = \mathbf{73{,}730\ kN}$$

The corrected base width B_3 at bottom of II strip

$$B_3 = \sqrt{\frac{\gamma_w^2 \cdot H_3^3}{f}\left[1 + \frac{\gamma_w^2 H_3^4}{4W_3^2}\right]}$$

$$B_3 = \sqrt{\frac{10 \times (96)^3}{3000}\left[1 + \frac{10^2 (96)^4}{4 \times (73{,}730)^2}\right]}$$

$$= \sqrt{2{,}949\,(1 + 0.39)} = \sqrt{4{,}100} = \mathbf{64.0\ m.}$$

Now X_3 is obtained from the equation [19.30 (*b*)], *i.e.*

$$\frac{\gamma_w \cdot S_s \cdot y_2}{24}\left[3B_2^2 - B_3^2 + 6X_3 (B_2 + B_3) + 2B_2B_3\right]$$

$$- \frac{\gamma_w \cdot X_3}{12}[H_2 + H_3][2B_3 - 3X_3] - W_2\left[\frac{B_3 - B_2}{3} - X_3\right] = 0$$

where $y_2 = 4.0$ m

$B_2 = 60.4$ m

$B_3 = 64.0$ m

$H_2 = 92$ m

$H_3 = 96$ m

$W_2 = 6{,}742$ tonnes

$\gamma_w = 10\text{ kN/m}^3$

Substituting, we get

$$\frac{10 \times 2.4 \times 4.0}{24}\left[3\,(60.4)^2 - (64.0)^2 + 6 \cdot X_3\,(60.4 + 64.0) + 2 + 60.4 \times 64.0\right]$$

$$- \frac{10 \cdot X_3}{12}\left[92 + 96\right]\left[2 \times 64.0 - 3X_3\right] - 67{,}420\left[\frac{64.0 - 60.4}{3} - X_3\right] = 0$$

or $0.4\,[10{,}920 - 4{,}100 + 747\,X_3 + 7{,}750] - X_3[2010 - 47X_3] = 6{,}742\,[1.20 - X_3] = 0.$

or $5{,}828 + 299X_3 - 2{,}010X_3 + 47X_3^2 - 8{,}091 + 6{,}742X_3 = 0$

or $47X_3^2 + 5{,}031X_3 - 2{,}263 = 0$

or $X_3^2 + 107X_3 - 48.2 = 0$

Solving this equation, we get, $\mathbf{X_3 = 0.45\ m}$

Hence, out of 64.2 m of base width the the base of II strip, 0.45 m shall be provided on u/s side as shown in Fig. 19.21 (*c*).

Design of last, *i.e.* third strip

Corrected weight of II strip

$$= 24 \times 1 \left[\frac{60.4 + 64.0}{2}\right] \times 4.0 = 5980 \text{ kN}$$

Weight of water resting on the u/s face of II strip

$$= 10\left[92 \times 0.45 + \frac{1}{2} \times 4 \times 0.45\right] = 500 \text{ kN}.$$

Total corrected weight at the base of II strip

$$W_3 = 67{,}420 + 5980 + 500 = \mathbf{73{,}900\ kN}$$

Depth of III strip = 4.0 m.

Approximate base width B_4' at the bottom of III strip

$$= 64.0 + \frac{56.8}{88.2} - 4.0 + 0.45$$

$$= 64.0 + 2.58 + 0.45 = 67.03 \text{ m}.$$

Approximate weight of III strip

$$= 24 \times 1 \left[\frac{64.0 + 67.03}{2}\right] \times 4.0 = 6280 \text{ kN}$$

Approximate weight of water resting on the u/s face of III strip if the u/s slope given to II strip is extended in III strip

$$= 10\left[\frac{96 + 100}{2} \times 0.45\right] = 442 \text{ kN}$$

Hence, the total approximate weight of dam and water at base of III strip

$$= W_4 = 73{,}980 + 6280 + 442 = 80{,}620 \text{ kN}$$

The corrected base width B_4 at the bottom of III strip is given by

$$B_2 = \sqrt{\frac{\gamma_w \cdot H_4^3}{f}\left[1 + \frac{\gamma_w^2 \cdot H_4^4}{4W_4^2}\right]}$$

where $f = 3000 \text{ kN/m}^2$

$H_4 = 100 \text{ m}$

$W_4 = 80{,}620 \text{ kN}.$

$$\therefore \quad B_4 = \sqrt{\frac{10 \times (100)^3}{3000}\left[1 + \frac{10^2 \times (100)^4}{4 \times (80620)^2}\right]}$$

$$= \sqrt{3{,}333\,[1 + 0.385]} = \sqrt{3{,}333\,(1.385)} = \sqrt{4{,}620} = 68.0 \text{ m}.$$

Now X_4 is obtained from the equation

$$\frac{\gamma_w \cdot S_s \cdot y_3}{24}\left[3B_3^2 - B_4^2 + 6X_4(B_3 + B_4) - 2B_3B_4\right]$$

$$- \frac{\gamma_w \cdot X_4}{12}\left[H_3 + H_4\right]\left[2B_4 - 3X_4\right] - W_3\left[\frac{B_4 - B_3}{3} - X_4\right] = 0,$$

where $y_3 = 4.0 \text{ m}$

$B_3 = 64.0 \text{ m}$

$$B_4 = 68.0 \text{ m}$$
$$H_3 = 96 \text{ m}$$
$$H_4 = 100 \text{ m}$$
$$W_3 = 7{,}390 \text{ tonnes.}$$

Substituting, we get

$$\frac{10 \times 2.4 \times 4.0}{24}\left[3\,(64.0)^2 - (68.0)^2 + 6 \cdot X_4\,(64.0 + 68.0) + 2 \times 64.0 \times 68.0\right]$$
$$-\frac{10 \cdot X_4}{12}\left[96 + 100\right]\left[2 \times 68.0 - 3X_4\right] - 73{,}900\left[\frac{68.0 - 64.0}{3} - X_4\right] = 0$$

or $4\,[12{,}288 - 4{,}624 + 792X_4 + 8.704] - 10X_4[2{,}220 - 49X_4] - 73{,}900\,[1.33 - X_4] = 0$

or $65{,}470 + 3170X_4 - 22{,}200X_4 + 490X_4^2 - 98{,}530 + 73{,}900X_4 = 0$

or $6{,}547 + 317x_4 - 2{,}220X_4 + 49X_4^2 - 9{,}853 + 7{,}390X_4 = 0$

or $49X_4^2 + 5{,}487X_4 - 3{,}406 = 0$

or $X_4^2 + 112X_4 - 69.5 = 0$

Solving, we get $\mathbf{X_4 = 0.62\ m.}$

Hence, out of base width of 68.0 m, at the base of III strip, 0.62 m shall be provided on the u/s side, and the remaining shall be provided on d/s side as shown in Fig. 19.21 (*c*).

The design of all the bottom strips is shown in Fig. 19.21 (*c*).

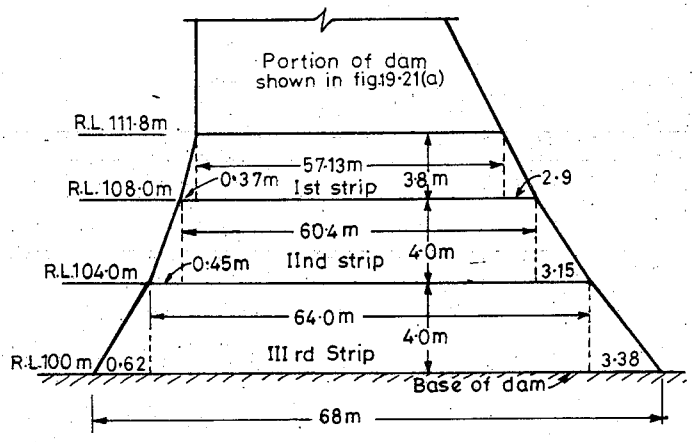


Fig. 19.21 (*c*)

CONSTRUCTION OF GRAVITY DAMS

19.11. Diversion Problem in Dams Construction

Before the actual construction of a dam can start in a river channel, the water of the river channel must be temporarily diverted. It is advantageous to schedule the construction of the lower portion of the dam during normal periods of low flow so as to minimise the diversion problem. The diversion of river water can be accomplished in either of the following two ways :

(*i*) **Provision of a Diversion Tunnel.** If geological and topographical conditions are favourable, a diversion tunnel or a diversion open channel may be constructed to carry the entire flow around the dam site as shown in Fig. 19.22. The area in which construction work has to take place, is closed by coffer-dams. The diversion tunnel or channel will start from upstream of the upstream coffer-dam and will join the river again on the downstream of the downstream coffer-dam, as shown in Fig. 19.22.

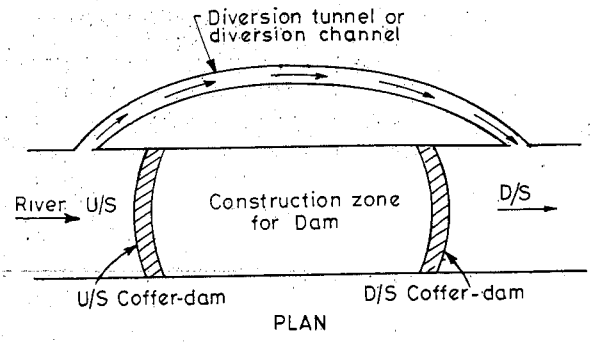


Fig. 19.22

(*ii*) **By constructing the dam in two stages.** The dam is sometimes constructed in two stages. In such a case, the flow is, first of all, diverted and confined to one side of the channel by constructing a semi-circular type of a coffer-dam as shown in Fig. 19.23. The construction work can be taken up in the water-free zone. When the work on the lower portion of the dam on half of its length in one side of the channel gets completed, the remaining half width of the channel is closed by a coffer-dam, as shown in Fig. 19.24. The flow is diverted

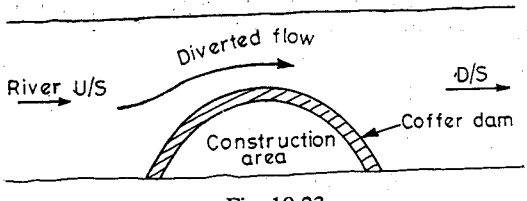


Fig. 19.23

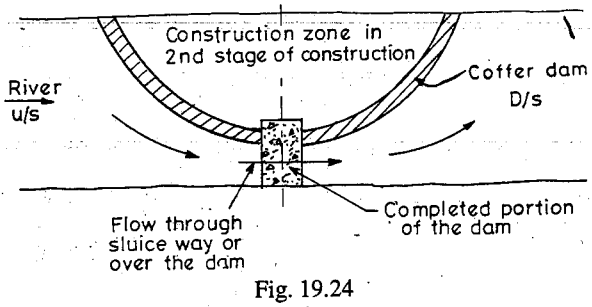


Fig. 19.24

through the dam outlets or sometimes it may even be allowed to overtop the already constructed portion of the dam. The work will continue in the water-free zone.

19.12. Construction of Galleries in Gravity Dams

Galleries are the horizontal or sloping openings or passages left in the body of the dam. They may run longitudinally (*i.e.* parallel to dam axis) or transversely (*i.e.* normal to dam axis) and are provided at various elevations. All the galleries are interconnected by steeply sloping passages or by vertical shafts fitted with stairs or mechanical lifts. The size of a gallery will depend upon the size of the dam and the function of the gallery.

Functions and Types of Galleries in Dams

(1) **Foundation Gallery.** A gallery provided in a dam may serve one particular purpose or more than one purpose. For example, a gallery provided near the rock

foundations, serves to drain off the water which percolates through the foundations. This gallery is called a *foundation gallery* or a *drainage gallery*. It runs longitudinally and is quite near to the upstream face of the dam. Its size usually varies from1.5 m × 2.2 m to 1.8 m × 2.4 m. Drain holes are drilled from the floor of this gallery after the foundation grouting has been completed. Seepage is collected through these drain holes. The size of the gallery should be sufficient to accommodate atleast a drilling machine. Besides draining off seepage water, it may be helpful for drilling and grouting of the foundations, when this can not be done from the surface of the dam.

(2) **Inspection Galleries.** The water which seeps through the body of the dam is collected by means of a system of galleries provided at various elevations (say at heights of 15 m or so) and interconnected by vertical shafts, etc. All these galleries, besides draining off seepage water, serve inspection purposes. They provide access to the interior of the dam and are, therefore, called *Inspection galleries*. However, galleries in dams are seldom provided for purely inspection purposes. They generally serve other purposes along with this purpose. Their main functions are summarised below :

(*i*) They intercept and drain off the water seeping through the dam body.

(*ii*) They provide access to dam interior for observing and controlling the behaviour of the dam.

(*iii*) They provide enough space for carrying pipes, etc. during artificial cooling of concrete, (see next article).

(*iv*) They provide access for grouting the contraction joints when this cannot be done from the face of the dam.

(*v*) They provide access to all the outlets and spillway gates, valves, etc. by housing their electrical and mechanical controls. All these gates, valves, etc. can hence, be easily controlled by men, from inside the dam itself.

(*vi*) They provide space for drilling and grouting of the foundations, then it cannot be done from the surface of the dam. Generally, the foundation gallery is used for this purpose.

Foundation gallery is generally differently named from inspection galleries, although strictly speaking, it can also be used for inspection purposes and may be called as inspection gallery.

19.12.1. Cross-sections of Dam Galleries. Dam galleries are formed as the concrete is placed and its size depends upon the function of the gallery and also upon the size of the dam. Certain important shapes of the commonly used galleries are shown below in Fig. 19.25.

The provision of a gallery in a dam body, changes the normal pattern of stresses in the body of the dam. Stress concentration may, therefore, occur at corners, and hence, in order to minimise this stress concentrations, the corners mast be rounded smoothly. Tension and compression zones may be worked out and proper reinforcements, etc. are provided to counteract them.

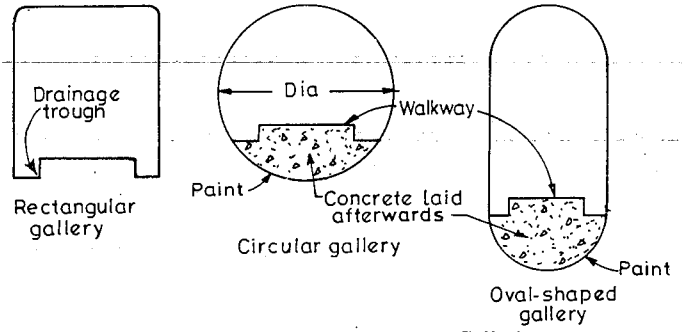


Fig. 19.25. Various Types of Dam Galleries.

19.13. Cracking of Concrete in Concrete Gravity Dams

When concrete sets, a tremendous amount of heat is liberated (due to heat of hydration of cement), which will raise the temperature inside the body of the dam. But the temperature outside the dam remains equal to the atmospheric temperature. Due to these temperature differences, temperature stresses get developed in the dam body. Besides, *due to shrinkage of concrete as it cools, shrinkage stresses get developed.* These temperature stresses and shrinkage stresses will cause the concrete to crack unless remedial measures are undertaken. Various measures, generally adopted in concrete gravity dams, to avoid this cracking are :

(*i*) Using minimum amount of cement in a given mix of specified strength. The quantity of cement can be decreased by better grading the aggregates.

(*ii*) 'Low lifts' should be used for concrete. When concrete is poured, it is poured up to a certain height in the first attempt. This height is called 'lift'. Generally, 1.5 m lift is used in modern dams. If this lift is reduced, more horizontal joints will get developed and also sufficient cooling time between two successive pours shall be obtained, thus reducing cracking.

(*iii*) By providing suitably spaced contraction joints, in addition to the normal construction joints.

(*iv*) Special low heat cements may be used.

(*v*) The materials which go into the concrete, may be cooled before mixing.

(*vi*) Further cooling is accomplished by circulating cold water through pipes embedded in concrete. This is quite an expensive measure and is adopted only for large gravity dams.

19.14. Joints in a Gravity Dam

The concreting of the dam is usually placed in blocks. The size of blocks will depend upon the size of dam and necessity of contraction joints required from the considerations of cracking of concrete. Their width is usually not more than 15 m or so, in large dams. Maximum height of a single pour of concrete (called 'lift') is usually about 1.5 m or so. The alternate blocks of the very first layer which is laid immediately over the rock foundation is taken as 0.75 m deep, as shown in Fig. 19.26. Dam sections are poured alternately so that each block is permitted to stand serveral days before another layer is poured on top of the first. Care must be taken to protect the individual dam sections from the drying action of air by ensuring full-fledged curing. After the shutterings are removed, the lateral surfaces of each section are painted with an asphaltic emulsion paint so as to prevent its adherence to the adjoining sections and thus to serve as construction joints to reduce cracking in concrete. The *Vertical joints* so developed are called **the transverse joints** and they run through the entire height and extend through the full width of the dam section. Transverse joints will thus divide the dam length into a number of vertical cantilevers each of which is independent of the other ; justifying the two dimensional analysis of gravity dams.

The *horizontal joints* called **longitudinal joints** are developed at each lift height and will extend through the entire width of the dam section. They shall run through the entire length of the dam but shall be staggered between transverse joints, as shown in Fig. 19.26.

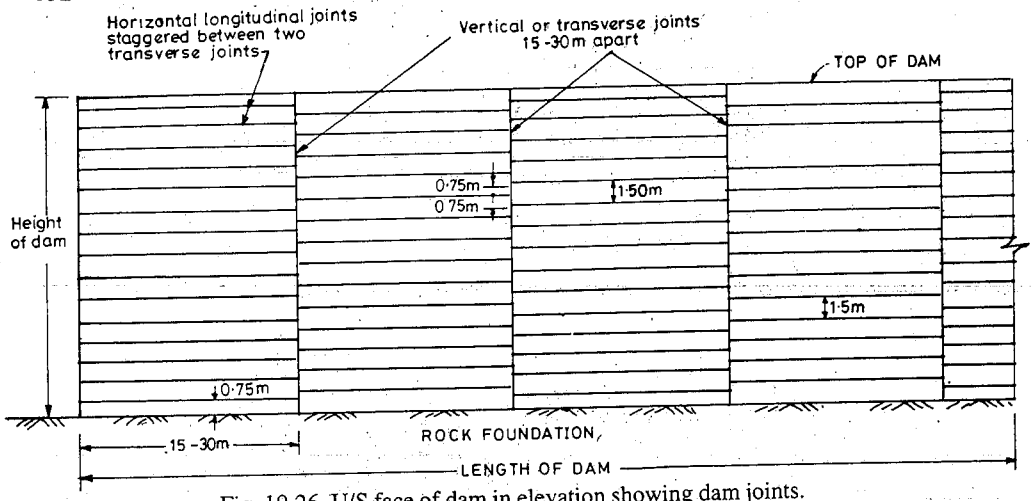


Fig. 19.26. U/S face of dam in elevation showing dam joints.

Construction joints and Contraction joints. Truly speaking, every joint whichsoever is left in the dam, shall be a construction joint and every *construction joint* will oppose contraction stresses, and hence will be a *contraction joint.* Therefore, there should be no difference between the two. But whether the joint left was needed from the considerations of practical difficulties in laying the concrete in a single stretch or it was left intentionally for making provisions for shrinkage and temperature stresses, sometimes defines the limits of these two terms. In other words, horizontal joints which were a must from considerations of lift, are sometimes called the construction joints ; while the joints which are mainly left for shrinkage of concrete are called the contraction joints. Although most of the times, these two terms are used to mean one and the same thing.

19.14.1. Shear Keys or Key ways.

Where foundation conditions are such that undesirable differential settlements or displacements between adjacent concrete blocks may occur, shear keys are formed in transverse vertical joints between sections to carry the shear from one section to the adjacent one and make the dam act as a monolithic structure. These shear keys between the transverse joints are seldom provided on good foundations and in U-shaped valleys, as they will unjustify the two dimensional design of the dam.

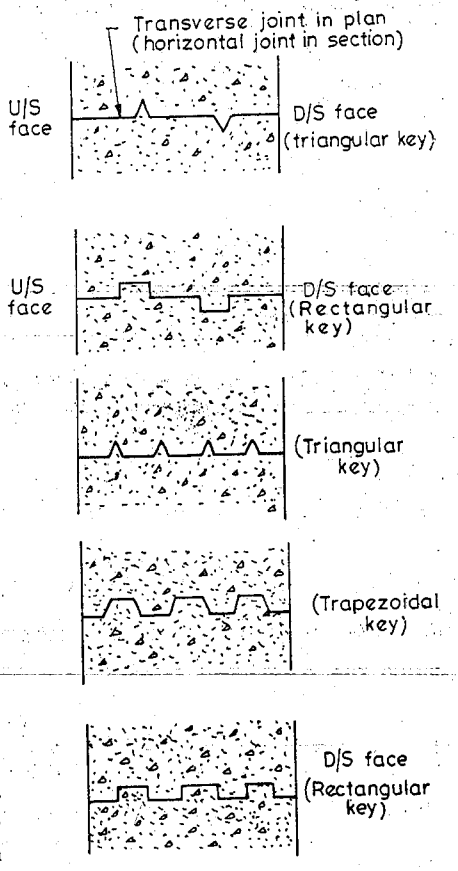


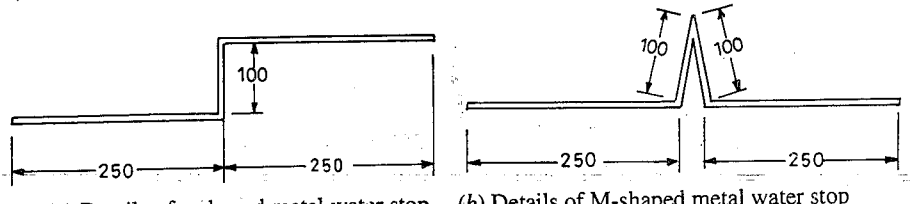
Fig. 19.27. Various types of key-ways.

Shear keys are however, always provided between the horizontal- longitudinal joints in order to transmit vertical shearing stresses across the section. After the concrete has undergone its shrinkage, these longitudinal horizontal joints are grouted through pipes embedded in the blocks before hand. The transverse vertical joints may also be keyed and grouted if the monolithic behaviour is desired as is generally done on poor foundations and in narrow V-shaped gorges. Various types of key ways used these days are shown in Fig. 19.27.

19.14.2. Water Stops. Water stops or water bars, as used in R.C.C. aqueducts (discussed in article 14.6), are also required to be provided in the transverse as well as horizontal joints in concrete adjacent to the upstream face of dam. The openings of the vertical transverse joints as well as of the longitudinal horizontal joints in a dam body (Fig. 19.26), if not sealed properly with water stops, will provide passage for seepage of water through the dam body. To stop this leakage, water stops consisting of metal strips, rubber or PVC are installed across the joints adjacent to the upstream face. *Asphalt water stops* are used as secondary water stops to the metal and rubber/PVC water stops.

In **metal water stops**, copper or stainless steel strips of not less than 1.5 mm thickness and 300 mm width are used. Z-shape and M-shape metal stops, as shown in Fig. 19.28 (*a*) and (*b*), are generally used.

Rubber water stops are made from natural rubber confirming to the requirements given in table 19.3. The shape of such water stops may vary, depending upon the water head, etc. The usual minimum width of rubber water stops for dams is 300 mm.



(*a*) Details of z-shaped metal water stop (*b*) Details of M-shaped metal water stop

Fig. 19.28. Various types of metal waterstops.

PVC waterstops should be fabricated from plastic compounds, the basin resin of which shall be polyvinyl chloride. The compound shall contain any additional resins, plasticizers, inhibitors or other material such that when the material is compounded, it shall meet the requirements given in table 19.3.

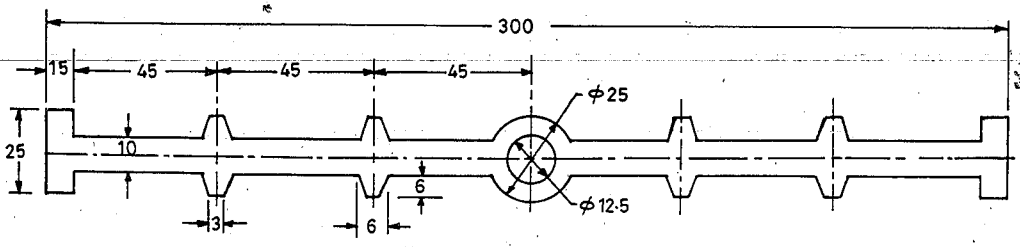
Table 19.3. Performance Requirements of Rubber or PVC Water Stops

S. No.	*Characteristics*	*Unit*	*Values*
(*i*)	Tensile strength	N/mm^2	11.5, min
(*ii*)	Ultimate strength	%	80, min
(*iii*)	Tear resistance	N/mm^2	4.9, min
(*iv*)	Stiffness in flexure	N/mm^2	2.46, min
(*v*)	Accelerated extraction		
	(*a*) Tensile strength	N/mm^2	10.5, min
	(*b*) Ultimate alongation	%	250, min
(*vi*)	Effect of alkali : 7 days		
	(*a*) Weight increase	%	0.10, max
	(*b*) Weight decrease	%	0.10, max.
	(*c*) Hardness change	Point	±5
(*vii*)	Effect of alkali : 28 days		
	(*a*) Weight increase	%	0.40, max
	(*b*) Weight decrease	%	0.90, max
	(*c*) Dimension change	%	±1

Certain typical shapes in which rubber/PVC water stops, suitable for concrete gravity dams, may be made are shown in Fig. 19.29. The shape recommended by IS : 12200-1987 for dams is shown in Fig. 19.30.

Shape of PVC or Rubber Waterstop	*Type*	*Category*	*Width*	*Thickness*	*Hydraulic Heads*	*Safe Stability*
(*i*)	230XA	Dumb Bell with Central Bulb	230	10	50	Used where large movement due to expansion/contraction is expected. Also for settlement joints and crumple sections
(*ii*)	230G	Serrated with Central bulb	230	4-5	25	For thicker concrete walls and with lower hydrostatic pressures for expansion joints.
(*iii*)	240H	Serrated with Central bulb	240	8-10	50	For thicker concrete wall and for higher hydrostatic pressure for expansion and construction joints.
(*iv*)	240RS	Ribbed and serrated	240	5	25	For construction joints
(*v*)	305F	Serrated with Central bulb	305	8-12	60	For very high hydrostatic pressure and where higher grading of concrete aggregates used for expansion joints in Dams etc.
(*vi*)	300N	Ribbed with Central bulb	300	10-12	60	Same as 305F but slightly thicker, specially designed for Narmada Project.
(*vii*)	230KD	Kicker	230		25	Externally placed where re-inforcement in R.C.C. elements congested. Used for construction joints.
(*viii*)	230KE	Kicker with Central bulb	230	5	25	Externally placed in R.C.C. elements congested. Used for movement joints.
(*ix*)	320KZ	Kicker	320	5	25	Surface waterstops. Used for construction joints of long lengths.

Fig. 19.29. Certain typical shapes in which rubber/PVC waterstops may be made to suit individual dam project.



All Dimensions in mm

Fig. 19.30. Detailed dimensions of a rubber/PVC waterstop recommended by IS : 12200-1987.

Metal and rubber/PVC water stops are often used together to reduce leakage through the joints in a dam as shown in Fig. 19.31 to 19.34.

An **Asphalt water stop** is constructed by forming a well of square opening across the contraction joints and filling the opening with asphaltic compound. The well may be fitted in advance with a steam pipe or an electrical heat conductor for reliquefying the asphalt. The asphalt to be used should confirm to the specifications shown in table 19.4 below.

Table 19.4. Recommended Specifications of Asphalt as per IS : 12200-1987 for Asphalt Joints

S. No.	*Property*	*Recommended Value*
1.	Density	1015–1065 kg/m^3
2.	Penetration at 25°C	200–300 mm
3.	Softening point (Ring and ball test)	80–90°C
4.	Brittleness test on 22 mm^2 specimen at 5°C, (energy absorbed)	0.97 kgm

The location, shape and dimensions of an asphalt water stop (square hole filled with asphalic compound) are shown within a typical sectional plan of a contraction joint in Fig. 19.31.

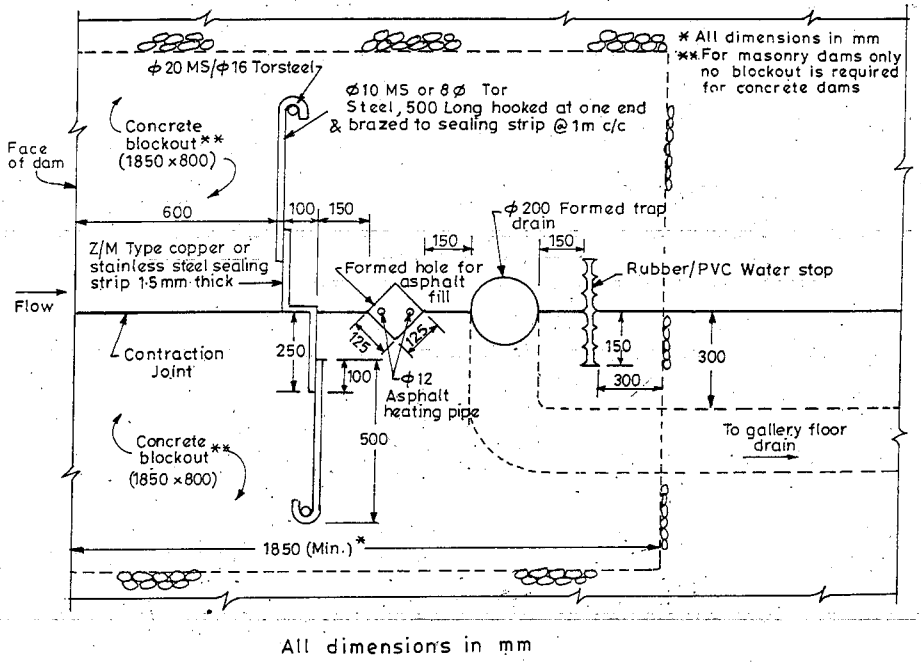


Fig. 19.31. Sectional plan at a contraction joint (square hole shows location of asphalt water stop)
Note : Dowel bars between concrete/masonry face of the blockout not shown.

19.14.2.1. Installation of water stops

(1) The metal waterstops shall be erected with the help of anchor rods.

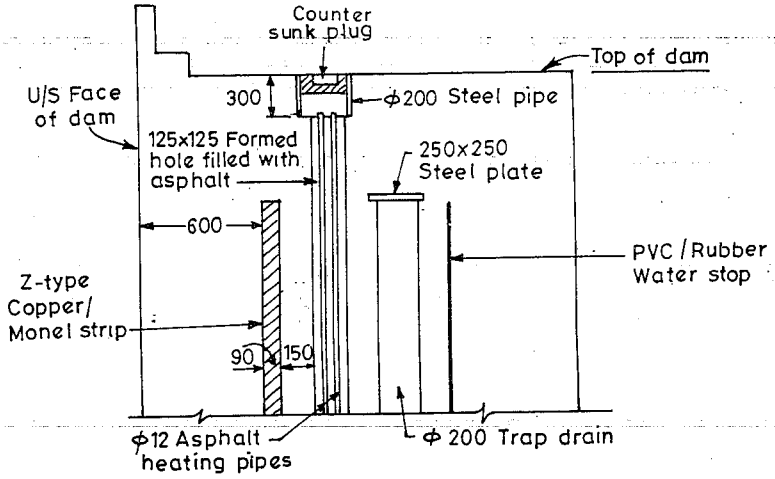
(2) In the case of masonry dams, the surface adjacent to the blockouts (shown by dotted lines in Fig. 19.30) shall be irregular and the joints in the masonry shall be raked out when mortar is green, with some stone protruding beyond dotted lines regularly in both directions. No such blockouts shall be provided in concrete dams where concreting on either side of the water seals is done along with the concreting of the rest of the concreting block.

(3) 25 mm dia dowel bars, 1500 mm long (500 mm in concrete and 1000 mm in masonry) 500 mm c/c in both directions shall be provided at the concrete/masonry interface of the blockout in case of a masonry dam to prevent shrinkage crack at the interface.

(4) The blockout may be concreted in lifts not more than 1.5 m. Minimum grade of concrete to be used in the blockout shall be M20.

(5) The blockout of one block may be concreted first and the joint face given a coat of coaltar black paint conforming to IS : 290-1961 and then only the blockout of the second block should be concreted, so as to have a clear contraction joint.

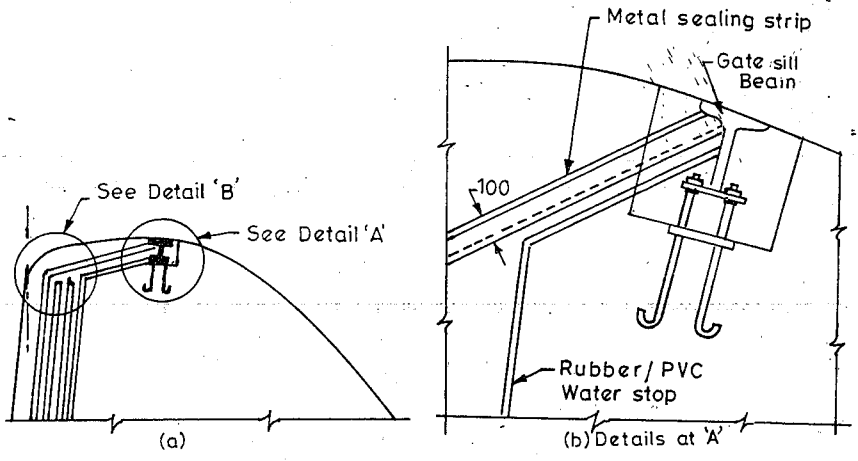
(6) Typical details of waterstop arrangement (at contraction joints between two monoliths of a dam) near the top of a non-overflow section are shown in Fig. 19.32 ; near the crest of an overflow section in Fig. 19.33 and near the bottom of the dam in Fig. 19.34.

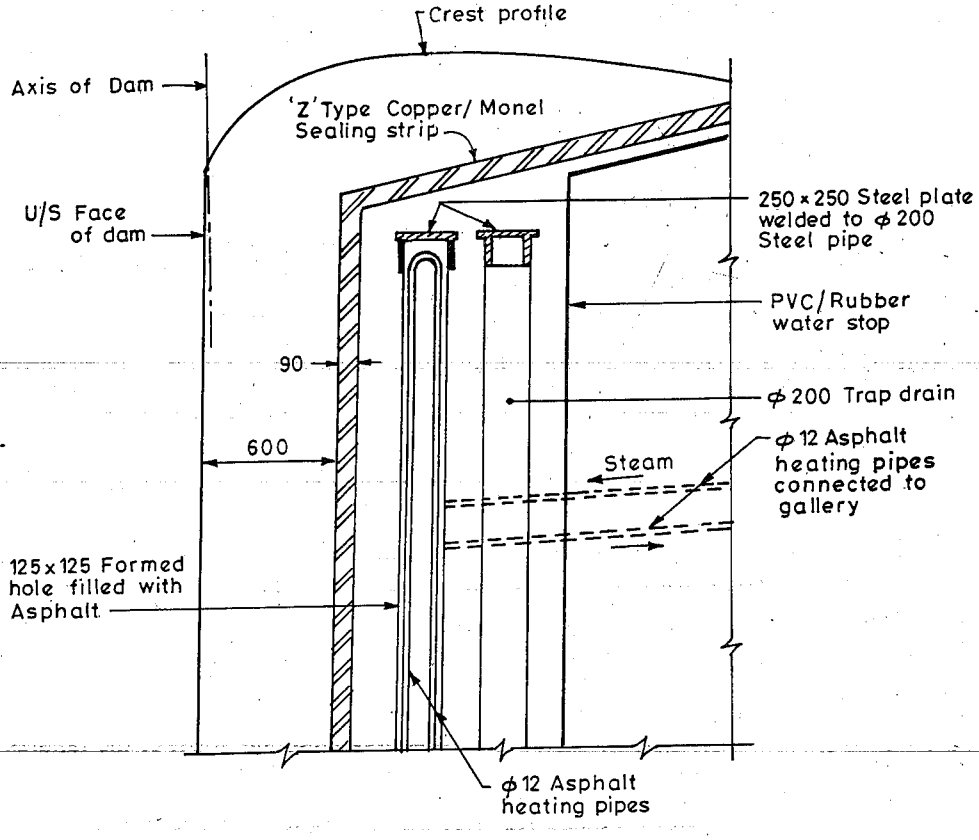


All Dimensions in mm

Fig. 19.32. Typical waterstop details near the top of non-overflow section of a dam.

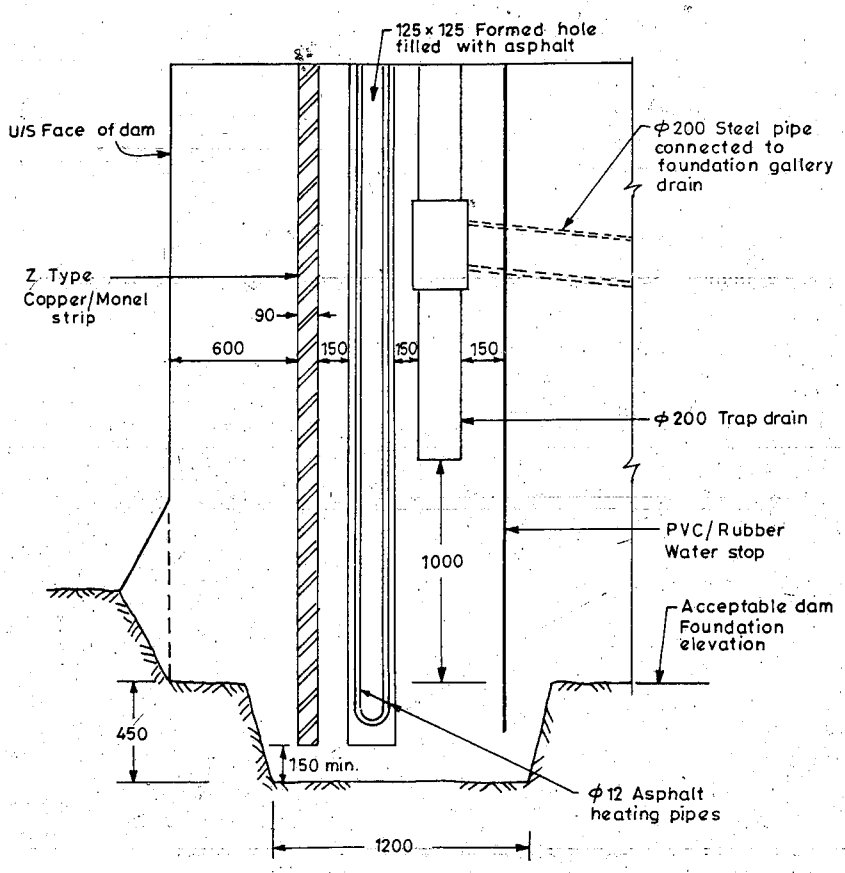
*IS code on "Methods of Testing Plastics".





(*c*) Details at 'B'
All dimensions in mm

Fig. 19.33. Typical waterstop details near the crest of overflow section (spillway).



All dimensions in mm

Fig. 19.34. Typical waterstop details near the bottom of the dam.

Rubber/PVC waterstops shall be provided around galleries/adits at the contraction joints between two monoliths of a dam, as shown in Fig. 19.35.

19.15. Foundation Treatment for Gravity Dams

The material underlying the base of a dam, *i.e.* the foundations of the dam, must be strong enough and capable to withstand the foundation pressure exerted on it under various conditions of loading and in dry as well as wet condition. Most of the failures of the dams have occurred because of the failure of their underlying strata. A concrete gravity dam of California called St. Francis dam was about 62 m high and about 210 m long and failed soon after its completion. The cause of failure was found to be the

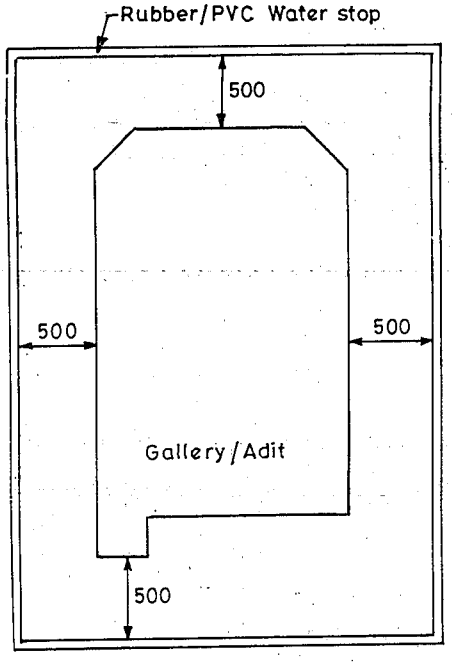


Fig. 19.35. Rubber/PVC waterstop around Dam Gallery/Adit at contraction joints.

presence of conglomerate in one abutment, which was weakened after exposure to moisture from the reservoir. Similarly, the failure of a 60 m high arch dam in France (called Malpasset dam) was attributed to the presence of a clay seam in the rock at one of the abutments. Austin Dam on the Colorado river in Texas failed in the year 1900, because large cavities had been dissolved in its limestone foundation.

All these examples have been quoted just to stress upon the readers and the designers, the importance of foundations and the need for their thorough investigation and remedial treatment, if any thing special is found adverse. Besides the special remedial measures in particular cases, the foundation treatment commonly adopted for all foundations can be divided into two steps :

(1) *Preparing the surface* ; and

(2) *Grouting the foundations.*

These treatments are briefly discussed below :

(1) **Preparing the Surface.** The surface preparation consists in removing the entire loose soil till a sound bed rock is exposed. The excavation should be carried out in such a way that the underlying rock is not damaged. The final surface obtained above is stepped, so as to increase the frictional resistance of the dam against sliding. The stepping of the foundation and provision of a shear key is shown in Fig. 19.36. The shear key may sometimes be provided in the centre but is generally provided at the heel.

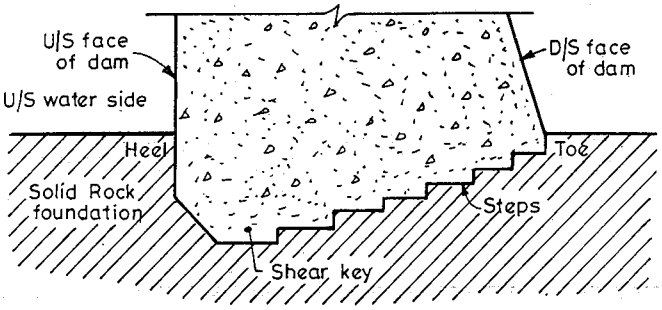


Fig. 19.36. Stepping of dam foundation and provision of shear key.

If faults, seams, or shattered rock zones are detected in the exploratory geological investigations, special steps and remedies must be taken to ensure their removal. They may have to be entirely excavated and back-filled with concrete grouting. The treatment will depend upon the specific needs.

The top foundation surface is thoroughly cleaned with wet sand blasting and washing before the concreting for dam section is started to be laid.

(2) **Grouting the Foundation.** The foundation grouting can be divided into :

(*a*) Consolidation grouting ; and

(*b*) Curtain grouting.

(*a*) *Consolidation grouting.* The entire foundation of the dam is consolidated by grouting. For this purpose, shallow holes (called *B* holes) are drilled through the foundation rock. The depths of these holes generally vary between 10 to 15 m. They are situated at about 5 to 20 m apart, in the general area of the heel of the dam. After the holes have been drilled, mixture of cement and water (called grout) is forced into the holes at low pressure of about 30 to 40 N/cm^2. This is accomplished, before any concreting for the dam section is laid. This low pressure grouting will result in a general consolidation of the foundations. These low pressure grout holes will later serve the purpose of a cut-off against leakage of high pressure grout, which is to be used after some concreting of the dam has taken taken place.

(*b*) *Curtain Grouting.* It helps in forming the principal barrier or a curtain against the seepage through the foundations, and thus reduce the uplift pressures. To accomplish this high pressure grouting, relatively deeper holes (called *A* holes) are drilled near the heel of the dam. The spacing of the holes may vary from 1.2 to 1.5 m. Holes are first of all, drilled and grouted at about 10 to 12 m apart, and then the intermediate holes are drilled and grouted. The depths of the holes vary from 30 to 40% of the total upstream water head for strong rock foundations, and may be as much as 70% of the water head for poor rocks. After the holes have been drilled, a mixture of cement and water (*i.e.* grout) is forced into the holes under high pressure. The grouting pressure may be kept as high as possible without lifting the foundation strata. Usually, the foundation pressure used in this high pressure grouting is equal to 2.5 D N/cm^2, where D is the depth of grouting in metres below the surface. This grouting is generally done in stages of depth equal to 15 m or so, and carried out only after some portion of the dam section has been laid.

This grouting may have to be accomplished from the foundation gallery or from other galleries within the dam. It may also be done from the upstream face of the dam, if possible. In certain special cases, this grouting may have to be accomplished from tunnels driven into the foundation rock below the dam.

PROBLEMS

1. (*a*) What is meant by gravity dams ?

(*b*) What are the main points to be considered while selecting a site for a gravity dam construction? *(Madras University, 1975)*

(*c*) Explain briefly with neat sketches the different forces that may act on a gravity dam. Indicate their magnitudes, directions and locations. *(Madras University, 1974)*

2. (*a*) Explain in details the various forces causing instability in a gravity dam. *(Madras University, 1974)*

(*b*) What is meant by the elementary profile of a gravity dam ; and how is it deduced ? What should be the maximum depth of elementary profile of a dam if the safe limit of stress on the masonry should not exceed 1500 kN per m^2 ? *(Madras University, 1975)*

[**Ans.** 44 m, assuming unit weight of masonry = 24 kN/m^3]

3. Explain briefly how grout curtain and drainage affect uplift pressures in gravity dams.

Fig. 19.37 shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate (neglecting earthquake effects):

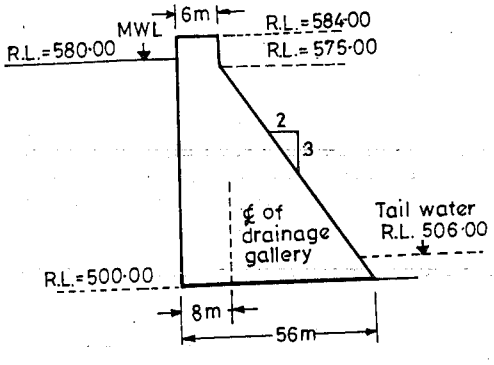


Fig. 19.37

(*i*) The maximum vertical stresses at the heel and toe of the dam.

(*ii*) The major principal stress at the toe of the dam.

(*iii*) The intensity of shear stress on a horizontal plane near the toe.

Assume weight of concrete = 24 kN/m^3.

[**Hint :** Follow example 19.1]

4. (*a*) What do you mean by the elementary profile of a gravity dam ? *(Madras University, 1976)*

(*b*) What are the advantages and disadvantages of a gravity dam over the other types ? *(Madras University, 1976)*

(*c*) What are the methods adopted to reduce uplift in masonry dams ? *(Madras University, 1976)*

5. (*a*) What is meant by the term 'low dam' ? Determine the dimensions of the elementary profile of a low gravity dam. *(Madras University, 1973)*

(*b*) Explain the criteria that govern the design of a high gravity dam in different zones of its cross-section. *(Madras University, 1976)*

6. (*a*) What is meant by "concrete gravity dams" ? Draw a neat typical cross-section of such a dam. Name the highest dam of the world as well as that of India.

(*b*) What are the different ways by which a concrete gravity dam may fail, and how will you ensure its safety against each type of failure ?

7. (*a*) Differentiate between a 'low gravity dam' and a 'high gravity dam'.

(*b*) How does the practical profile of a low gravity dam differs from that of the theoretical one, and why ?

(*c*) Discuss step by step the analytical procedure that you will adopt for analysing the stability (two dimensional analysis) of gravity dams.

8. Write detailed notes on any two of the following :

(*i*) Forces acting on gravity dams.

(*ii*) Stability analysis of gravity dams.

(*iii*) Elementary profile of a gravity dam.

(*iv*) Design considerations and fixing the section of a gravity dam.

(*v*) Design criteria for the design of high gravity dams.

9. Write short notes on :

(*i*) Uplift force.

(*ii*) Drainage gallery.

(*iii*) Grout curtain.

(*iv*) Construction joint.

(*v*) Earthquake forces on dams. *(Madras University, 1973)*

10. (*a*) Derive an expression for the limiting height of a low dam. *(Madras University, 1974)*

(*b*) Briefly explain the functions of the following :

(*i*) Drainage gallery.

(*ii*) Construction joints in a dam.

(*iii*) Ogee spillway. *(Madras University, 1974)*

[**Hint :** For (*iii*) Please refer chapter 21]

11. A concrete dam can be assumed to be trapezoidal in section having a top width of 2 m and bottom width of 10 m. Its height is 12 m and the upstream face has a batter of 1 : 10. Give an analysis of the stability of the dam for the base section for overturning and sliding in the full reservoir condition assuming no free-board allowance but allowing for uplift pressures. Assume uplift intensity factor as 100%. Also determine the compressive stresses at the toe and the heel, and major principal and shear stress developed at the toe. Assume weight of concrete to be 24 kN/m^3, unit shear strength of concrete to be 1400 kN/m^3, and the coefficient of friction between concrete and foundation soil to be 0.7.

12. The following data refer to the non-overflow section of a gravity dam :

R.L. of top of the dam	= 315 m
R.L. of bottom of the dam	= 260 m
Full reservoir level	= 312 m
Top width of the dam	= 12 m.

Upstream face is vertical. Downstream face is vertical upto R.L. 304 m ; and thereafter, the downstream face slopes at 0.7 (*H*) : 1 (*V*) up to base.

Drainage holes are located 8 m away from the upstream face.

Unit weight of masonry = 23 kN/m^3

Reduction of uplift at drainage hole = 50%

Coefficient of friction between masonry and foundation material = 0.8.

Determine (*i*) factor of safety against overturning ; (*ii*) factor of safety against sliding ; (*iii*) maximum pressure on foundation, and (*iv*) maximum principal stress in the masonry of the dam, at the

base. Consider only the forces due to water thrust, uplift, earthquake (inertial forces due to weight of masonry only) and the self-weight.

13. (*a*) Discuss the evolution of the final profile of a gravity dam from its elementary triangular profile, and explain the main principles of its design.

(*b*) Write a brief note on the necessity and method of foundation treatment of dams.

14. (*a*) Explain how uplift considerations affect the design of gravity dams. What measures can be adopted to reduce the undesirable effects due to uplift in such cases ?

(*b*) Write short notes on :

(*i*) Dam galleries.

(*ii*) Cracking of concrete during the construction of concrete gravity dams, and remedial measures.

(*iii*) Provision of keyways in concrete gravity dams.

15. (*a*) Differentiate between rigid dams and non-rigid dams.

(*b*) How are pore pressures and uplift pressures controlled in rigid dams ?

(*c*) Describe briefly the investigations that are necessary, and the treatment which is commonly given to the foundations of gravity dams, so as to avoid their failures in general.

16. The cross-section of a low gravity dam is shown in Fig. 19.38.

Assuming the reservoir to be full, determine

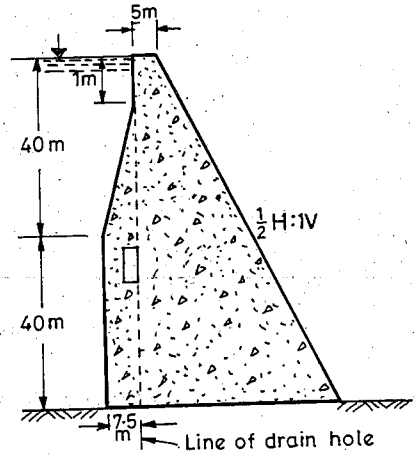


Fig. 19.38

(*i*) The normal stress ;

(*ii*) The principal stress ;

(*iii*) The shear friction factor at base.

Count full uplift as per U.S.B.R. recommendations. Neglect earthquake forces, wave pressure and silt pressure.

17. Design the practical profile of a gravity dam made of stone masonry given the following data :

R.L. of base of dam = 198 m.

R.L. of HFL of reservoir = 228 m.

Specific gravity of masonry = 2.4.

Safe compressive stress in masonry = 1200 kN/m^2.

18. Fig. 19.39 shows the section of a concrete gravity dam. Check the stability of this dam section at the base. Assume any data not given and needed.

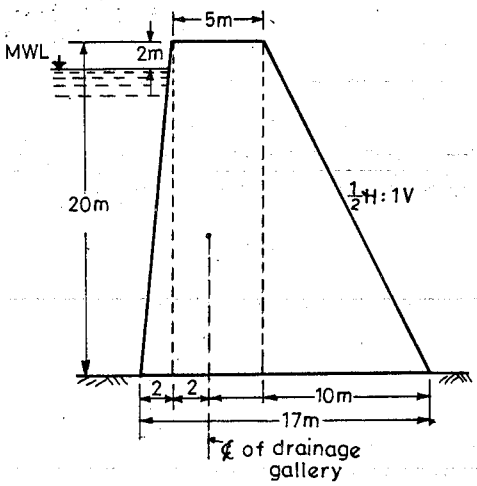


Fig. 19.39

20

Earthen Dams and Rock Fill Dams

20.1. Introduction

Earthen dams and earthen levees are the most ancient type of embankments, as they can be built with the natural materials with a minimum of processing and primitive equipment. But in ancient days, the cost of carriage and dumping of the dam materials was quite high. However, the modern developments in earth moving equipments have considerably reduced the cost of carriage and laying of the dam materials. The cost of gravity dams on the other hand, has gone up because of an increase in the cost of concrete, masonry, etc. Earthen dams are still cheaper as they can utilise the locally available materials, and less skilled labour is required for them.

Gravity dams and arch dams require sound rock foundations, but earthen dams can be easily constructed on earth foundations. However, earth dams are more susceptible to failure as compared to rigid gravity dams or arch dams. Before the development of the subject of Soil-Mechanics, these dams were being designed and constructed on the basis of experience, as no rational basis for their design was available. This led to the failure of various such earthen embankments. However, in these days, these dams can be designed with a fair degree of theoretical accuracy, provided the properties of the soil placed in the dam, are properly controlled. This condition makes the design and construction of such dams, thoroughly interdependent. Continuous field observations of deformations and pore water pressures have to be made during the construction of such dams. Suitable modifications in the design, are then made during construction, depending upon these field observations.

20.2. Types of Earthen Dams

The earthen dam can be of the following three types :

1. *Homogeneous Embankment type*
2. *Zoned Embankment type*
3. *Diaphragm type.*

These three types of dams are described below :

(1) **Homogeneous Embankment Type.** The simplest type of an earthen embankment consists of a single material and is homogeneous throughout. Sometimes, a blanket of relatively impervious material may be placed on the upstream face. A purely homogeneous section is used, when only one type of material is economically or locally available. Such a section is used

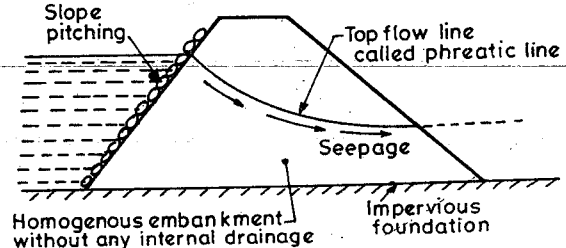


Fig. 20.1 (*a*). Homogeneous type embankment.

for low to moderately high dams and for levees. Large dams are seldom designed as homogeneous embankments.

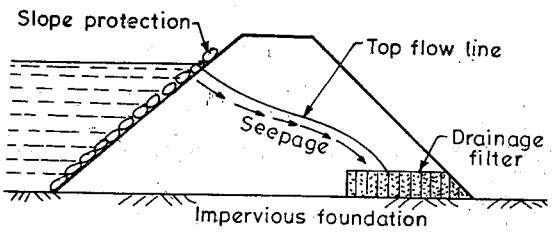


Fig. 20.1 (*b*). Homogeneous embankment provided with a drainage filter.

A purely homogeneous section poses the problems of seepage, and huge sections are required to make it safe against piping, stability, etc. Due to this, a homogeneous section is generally added with an internal drainage system : such as a horizontal drainage filter [Fig. 20.1 (*b*)], rock toe, etc. The internal drainage system keeps the phreatic line (*i.e.* top seepage line) well within the body of the dam, and steeper slopes and thus, smaller sections can be used. the internal drainage is, therefore, always provided in almost all types of embankments.

(2) **Zoned Embankment Type.** Zoned embankments are usually provided with a central impervious core, covered by a comparatively pervious transition zone, which is finally surrounded by a much more pervious outer zone (Fig. 20.2).

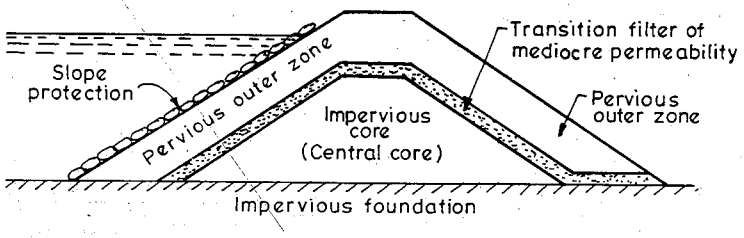


Fig. 20.2. Zoned type embankment.

The central core checks the seepage. The transition zone prevents piping through cracks which may develop in the core. The outer zone gives stability to the central impervious fill and also distribute the load over a large area of foundations.

This type of embankments are widely constructed and the materials of the zones are selected depending upon their availabilities. *Clay, inspite of it being highly impervious, may not make the best core, if it shrinks and swells too much. Due to this reason, clay is sometimes mixed with fine sand or fine gravel, so as to use it as the most suitable material for the central impervious core. Silts or silty clays may be used as the satisfactory central core materials. Freely draining materials, such as coarse sands and gravels, are used in the outer shell.* Transition filters are provided between the inner zone and the outer zone, as shown in Fig. 20.2. This type of transition filters are always provided, whenever there is an abrupt change of permeability from one zone to the other.

(3) **Diaphragm Type Embankments.** Diaphragm type embankments have a thin impervious core, which is surrounded by earth or rock fill. The impervious core, called diaphragm, is made of impervious soils, concrete, steel, timber or any other material. It acts as a water barrier to prevent seepage through the dam. The diaphragm may be placed either at the centre as a central vertical core or at the upstream face as a blanket. The diaphragm must also be tied to the bed rock or to a very impervious foundation material, if excessive under-seepage through the existing previous foundations has to be avoided (Fig. 20.3).

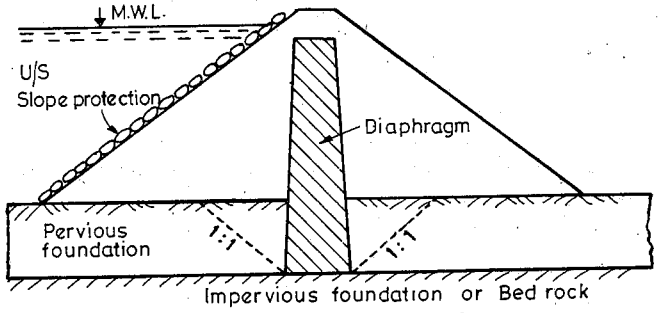


Fig. 20.3. Diaphragm type embankment.

The diaphragm type of embankments are differentiated from zoned embankments, depending upon the thickness of the core. If the thickness of the diaphragm at any elevation is less than 10 metres or less than the height of the embankment above the corresponding elevation, the dam embankment is considered to be of 'Diaphragm Type'. But if the thickness equals or exceeds these limits, it is considered to be of zoned embankment type.

20.3. Methods of Construction

There are two methods of constructing earthen dams :

(1) *Hydraulic-fill Method* ; and

(2) *Rolled-fill Method.*

(1) **Hydraulic-fill Method.** In this method of construction, the dam body is constructed by excavating and transporting soils by using water. Pipes called flumes, are laid along the outer edge of the embankment. The soil materials are mixed with water and pumped into these flumes. The slush is discharged through the outlets in the flumes at suitable intervals along their lengths. The slush, flowing towards the centre of the bank, tends to settle down. The coarser particles get deposited soon after the discharge near the outer edge, while the fines get carried and settle at the centre, forming a zoned embankment having a relatively impervious central core.

Since the fill is saturated when placed, high pore pressures develop in the core material, and the stability of the dam must be checked for these pressures. This type of embankment is susceptible to settlement over long periods, because of slow drainage from the core.

Hydraulic-fill method is, therefore, seldom adopted these days. Rolled-fill method for constructing earthen dams is, therefore, generally and universally adopted in these modern days.

(2) **Rolled-fill Method.** The embankment is constructed by placing suitable soil materials in thin layers (15 to 30 cm) and compacting them with rollers. The soil is brought to the site from burrow pits and spread by bulldozers, etc. in layers. These layers are thoroughly compacted by rollers of designed weights. Ordinary road rollers can be used for low embankments (such as for levees or bunds) ; while power-operated rollers are to be used for dams. The moisture content of the soil fill must be properly controlled. The best compaction can be obtained at a moisture content somewhere near the optimum moisture content. (The optimum moisture content is the moisture required for obtaining optimum density in the fill). Compaction of coarse gravels cannot be properly done by rolling and is best done by vibrating equipment. Detail of rolling and compactiing different types of soils are available in "Soil Mechanics and Foundation Engineering" by the same author.

20.4. Shearing Strength of Soils

Before we describe the causes of failures of earthen dams and the criteria for their safe design ; we shall review a few important conceptions of soil-mechanics and their importance in the design of earth dams.

Coulomb's Law. The resistance offered by a soil mass against the shearing forces, is known as the shear strength of the soil. In order to ensure that the foundation soil and the soil of the body of the dam are stable, we must ensure that the shearing strength of this soil mass is sufficiently greater than the shear stresses developed within the soil mass by external forces. According to Coulomb's Law, the shear strength of any soil mass is provided by two factors, *i.e.*

(*i*) Cohesion between the soil grains.

(*ii*) Internal friction between the soil particles.

Thus, the unit shear strength (τ_f) of a soil is given by

$$\tau_f = c + \sigma' \cdot \tan \phi \ (\text{kN/m}^2) \qquad ...(20.1)$$

where c = Unit cohesion of soil in (kN/m^2)

σ' = intergranular compressive stress, called the effective vertical stress (kN/m^2)

ϕ = Angle of internal friction of soil.

The constant c and ϕ depend upon the type of soil and the moisture content in it. Sands and gravels are cohesionless soils and $c \approx 0$ for them ; while ϕ is approximatrely zero for saturated cohesive impervious clays.

20.5. Various Kinds of Densities and Their Relations

Every soil has pores or voids in it, which remain filled either with air or with water or with both, depending upon the void ratio (e) and the moisture content (w) of the soil. The void ratio, porosity, moisture content, etc. of a soil are defined as follows :

$$\text{Void ratio } (e) = \frac{\text{Volume of voids in a soil mass}}{\text{Volume of solids in the soil mass}} = \frac{V_v}{V_s}$$

$$\text{Porosity of soil } (n) = \frac{\text{Volume of voids}}{\text{Total volume of soil}} = \frac{V_v}{V}$$

$$= \frac{V_v}{V_s + V_v} = \frac{\frac{V_v}{V_s}}{1 + \frac{V_v}{V_s}} = \frac{e}{1+e}$$

or $$e = \frac{1+e}{n}$$

$$\text{or Water content of soil } (w) = \frac{\text{Mass of moisture in soil mass}}{\text{Mass of solids in soil mass}}$$

$$= \frac{M_w}{M_s}$$

where M_w = Mass of water in the soil mass.

M_s = Mass of solids in the soil mass.

The unit weight of soil (γ) called bulk unit weight, is defined as the weight per unit volume of soil mass. If W is the total weight of soil, then

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_v} \text{ (air does not have any weight)}$$

$$\gamma = \frac{W_s\left(1+\frac{W_w}{W_s}\right)}{V_s\left(1+\frac{V_v}{V_s}\right)} \quad \text{...(A)}$$

or

Now $$\frac{W_s}{V_s} = \gamma_s = S_s \cdot \gamma_w$$

where γ_s = unit weight of solids

γ_w = unit weight of water, and

S_s = specific gravity of solid soil particles $= \frac{\gamma_s}{\gamma_w}$

Substituting in (A), we get

$$\therefore \quad \gamma = \frac{W_s}{V_s}\left[\frac{1+\frac{W_w}{W_s}}{1+\frac{V_v}{V_s}}\right] = S_s \cdot \gamma_w \left[\frac{1+w}{1+e}\right]$$

Hence, $$\gamma = S_s \cdot \gamma_w \cdot \left[\frac{1+w}{1+e}\right] \quad \text{...(20.2)}$$

If the soil mass is dry, $w = 0$

$$\therefore \quad \gamma_{dry} = \frac{S_s \cdot \gamma_w}{1+e} \quad \text{...(20.3)}$$

$$\therefore \quad \gamma = \gamma_{dry}(1+w)$$

Degree of saturation (S) is defined as

$$S = \frac{V_w}{V_v} = \frac{\text{Volume of water}}{\text{Volume of voids}}$$

Now $$e = \frac{V_v}{V_s} = \frac{V_v}{V_s} \times \frac{V_w}{V_w} = \frac{V_v}{V_w} \times \frac{V_w}{V_v}$$

$$= \frac{1}{S}\left[\frac{\frac{M_w}{\gamma_w}}{\frac{M_s}{S_s \cdot \gamma_w}}\right] = \frac{1}{S} \cdot \frac{M_w \cdot S_s}{M_s} = \frac{1}{S} \cdot w \cdot S_s$$

$$\boxed{e = \frac{wS_s}{S}} \quad \text{...(20.4)}$$

For saturated soils $S = 1$,

$$\therefore \quad e = w \cdot S_s$$

$$\therefore \quad \boxed{\text{Moisture content, } w = \frac{e}{S_s} \text{ for saturated soil}} \quad \text{...(20.5)}$$

Now, if unit weight of saturated soil is designated as γ_{sat}, then

$$\gamma = \frac{S_s\gamma_w(1+w)}{(1+e)} \quad \text{from Eq. (20.2)}$$

$$\gamma_{sat} = \frac{S_s \cdot \gamma_w \left(1 + \frac{e}{S_s}\right)}{1+e} = \frac{S_s \cdot \gamma_w (S_s + e)}{S_s (1+e)}$$

or
$$\boxed{\gamma_{sat} = \frac{\gamma_w (S_s + e)}{(1+e)}} \qquad ...(20.6)$$

Submerged unit weight of soil is defined as the unit weight of saturated soil minus the unit weight of water, and is designated as γ_{sub}.

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = \frac{\gamma_w (S_s + e)}{1+e} - \gamma_w$$

$$\gamma_{sub} = \gamma_w \left[\frac{S_s - 1}{1+e}\right] \qquad ...(20.7)$$

20.6. Pore-Water Pressure and its Significance in the Design of Earth Dams

We know that every soil has some voids or pores which are partly or fully filled with water. Let us consider a soil mass below the water-table *BB* in Fig. 20.4.

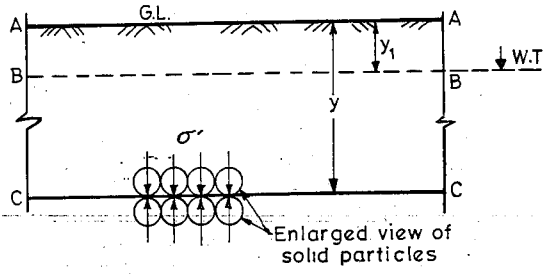


Fig. 20.4

The soil below *BB* is fully saturated and all its pores between the solid particles of soil, are full of water. The soil below the water-table is, therefore, subjected to hydrostatic uplift. Hence, at any level (say *CC*), the total downward normal pressure (σ) exerted by the weight of the soil above this level, shall be supported partly by the intergranular pressure (σ' or σ_{eff}) developed between dry particles of the soil, and partly by the hydrostatic pressure (u) due to the water present in the pores. Hence

$$\sigma = \sigma' + u \qquad ...(20.8)$$

where, σ is the total normal pressure on soil.

σ' = Total effective pressure, *i.e.* the intergranular pressure, or the pressure which is transmitted from grain to grain of soil.

u = Pore water pressure or *neutral pressure*. (It is the hydrostatic pressure due to presence of water in the soil pores.)

In Fig. 20.4, if the pressures are considered at a level say *CC* at a depth y from the ground, we can easily write,

Total normal pressure (σ) at level *CC*

$$= \gamma_{dry}\, y_1 + \gamma_{sat}\, (y - y_1)$$

where y_1 is the depth of watertable below the ground level.

$$\therefore \quad \sigma = \gamma_{dry}\, y_1 + \gamma_{sat.}\, (y - y_1)$$

Pore water pressure $= u = \gamma_w\, (y - y_1)$

Hence, the net effective pressure (σ') is given by

$$\sigma' = \sigma - u = \gamma_{dry}\, y_1 + \gamma_{sat}\, (y - y_1) - \gamma_w\, (y - y_1)$$
$$= \gamma_{dry}\, y_1 + \gamma_{sub}\, (y - y_1)$$

It, therefore, becomes evident that the effective normal stress is much less than the total normal stress, as a part of the total stress gets consumed by water as pore pressure. The effective stress is dependent on the submerged unit weight of soil.

The water pressure or pore pressure acts equally in all directions. It does not press the soil grains against one another, and, therefore, does not lead to compression of the soil or an increase in its frictional resistance, that is why, it is called a 'neutral stress'.

When the pore pressure is considered, the Coulomb's law will become

Unit Shear Strength of soil $(\tau_f) = c + \sigma' \cdot \tan \phi$

or
$$\boxed{\tau_f = c + (\sigma - u) \tan \phi} \qquad ...(20.9)$$

This clearly indicates that the shearing strength of a soil gets reduced due to the presence of pore pressure. The pore water pressure gets developed in the body of the earthen dam when the seepage takes place through the body of the dam, thus reducing the shear strength of the soil.

The shear stress (τ) developed at any plane in an earth structure is given by

$$\tau = \frac{\sigma_1 - \sigma_2}{2} \cdot \sin 2\theta \qquad ...(20.10)$$

where σ_1 = major principal stress
σ_2 = minor principal stress
θ = · is the angle between the plane considered for shear and the plane on which σ_1 acts.

It is apparent from the Equation (20.10), that the value of shearing stress remains unaltered, whether σ_1 and σ_2 are used or their effective components $[(\sigma_1 - u)$ and $(\sigma_2 - u)]$ are used. But the shear strength of the soil gets reduced when effective component $(\sigma - u)$ is used in place of σ.

Hence, the stability of the dam against shear failure must be checked when the maximum pore water pressure is present

Consolidation. The development of pore pressure is important, even during the time of construction of an earth dam. When the fully or partly saturated soil is placed and rolled in the body of the dam, the entire applied external load is taken up by the water immediately, and transferred to the soil afterwards. The pore water thus gets compressed, and if it is unable to drain out freely (due to low permeability of the soil), the pore pressure rises. *This rise in pore pressure during compaction is known as* **'hydrostatic excess' pressure in pore water.** It further reduces the shearing strength of the soil and hence, the stability of the soil. As the excess water drains out, more and more consolidation will take place, as this pressure will be transferred to soil grains and the

shear strength will tend to achieve its normal value. *Hence, the pore pressure temporarily reduces the shear strength of the soil during compaction* by preventing full compaction. But the shear strength gets recovered after the compaction is over, as the pore water is ultimately squeezed out.

In highly compressible soils, having low coefficient of permeability and moisture content above its optimum moisture content, this condition of 'hydrostatic excess' becomes very serious. Such soils are, therefore, more liable to fail during construction.

Similarly, appreciable consolidation of soil may takes place in fine grained compressible soils like clay, even after the construction is over, though sufficient compaction was done during construction.

Due to these reasons, pore pressure observations are often made during the construction period of an earth dam. If the hydrostatic excess of pore pressure rises to a dangerous amount, the construction may be stopped for some time till the excess water drains out and full natural compaction takes place. The construction may be restarted after this 'excess' is either fully dissipated or reduced to a safe value. The shear failure of the soil of the dam or its foundation is, therefore, very much connected with the development of pore pressure in the body of the dam and in the foundation, and must be properly checked and accounted for.

20.7. Causes of Failure of Earthen Dams

Earth dams are less rigid and hence more susceptible to failure. Every past failure of such a dam has contributed to an increase in the knowledge of the earth dam designers. Earthen dams may fail, like other engineering structures, due to improper designs, faulty constructions, lack of maintenance, etc. The various causes leading to the failure of earth dams can be grouped into the following three classes :

(1) Hydraulic failures
(2) Seepage failures
(3) Structural failures.

These causes are described below in details :

20.7.1. Hydraulic failures. About 40% of earth dam failures have been attributed to these causes. The failure under this category, may occur due to the following reasons:

(*a*) *By over topping.* The water may overtop the dam, if the design flood is underestimated or if the spillway is of insufficient capacity or if the spillway gates are not properly operated. Sufficient freeboard should, therefore, be provided as an additional satety measure.

(*b*) *Erosion of upstream face.* The waves developed near the top water surface due to the winds, try to notch-out the soil from the upstream face and may even, sometimes, cause the slip of the upstream slope. Upstream stone pitching or riprap should, therefore, be provided to avoid such failures.

(*c*) *Cracking due to frost action.* Frost in the upper portion of the dam may cause heaving and cracking of the soil with dangerous seepage and consequent failure. An additional freeboard allowance upto a maximum of say 1.5 m should, therefore, be provided for dams in areas of low temperatures.

(*d*) *Erosion of downstream face by gully formation.* Heavy rains falling directly over the downstream face and the erosive action of the moving water, may lead to the formation of gullies on the downstream face, ultimately leading to the dam failure. This can be avoided by proper maintenance, filling the cuts from time to time especially

during rainy season, by grassing the slopes and by providing proper berms at suitable heights (Fig. 20.5), so that the water has not to flow for considerable distances. The proper drainage arrangements are made for the removal of the rain water collected on the horizontal berms. Since the provision of berms ensures the collection and removal of water before it acquires high downward velocities, the consequent erosion caused by the moving water (run off) is considerably reduced.

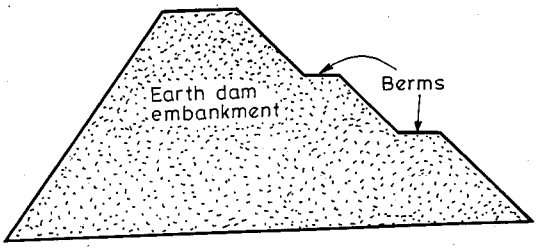


Fig. 20.5

(*e*) *Erosion of the d/s toe.* The d/s toe of the earth dam may get eroded due to two reasons, *i.e.* (*i*) the erosion due to cross currents that may come from the spillway buckets; and (*ii*) the erosion due to tail water. This erosion of the toe can be avoided by providing a downstream slope pitching or a riprap up to a height slightly above the normal tail water depth. Side walls of the spillway (called diaphragm walls) must be of sufficient height and length, as so to prevent the possibility of the cross flow towards the earthen embankment.

20.7.2. Seepage Failures. Controlled seepage or limited uniform seepage is inevitable in all earth dams, and ordinarily it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead to piping or sloughing and the subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam. Sloughing is the progressive removal of soil from the wet downstream face. More than 1/3rd of the earth dams have failed because of these reasons.

(*a*) *Piping through foundations.* Sometimes, when highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation of the dam, water may start seeping at a huge rate through them (Fig. 20.6). This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately resulting in a rush of water and soil, thereby creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure.

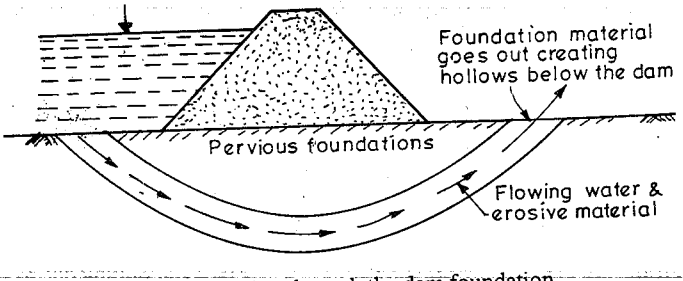


Fig. 20.6. Piping through the dam foundation.

(*b*) *Piping through the dam body.* When the concentrated flow channels get developed in the body of the dam, (Fig. 20.7) soil may be removed in the same manner as was explained in foundation piping, leading to the formation of hollows in the dam body, and subsequent subsidence of the dam. These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to

foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better maintenance of the dam embankments.

Piping through the dam body, generally get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits may develop into piping. This can be avoided by thoroughly and properly compacting the soils near the outlet conduits and by preventing the possibilities of leakage through conduits, but preventing the formation of cracks in the conduits. These cracks in the conduits are caused by differential settlement and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.

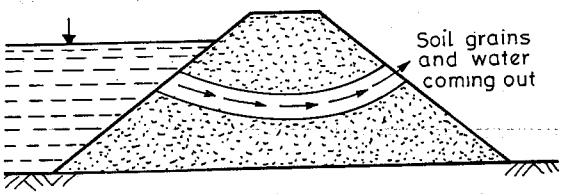


Fig. 20.7. Piping through the dam body.

(*c*) *Sloughing of D/S Toe.* The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and get eroded, producing a small slump or a miniature slide. The miniature slide leaves a relatively steep face which becomes saturated by the seepage from the reservoir and slumps again, forming a more unstable surface. The process continues till the remaining portion of the dam is too thin to withstand the horizontal water pressure, leading to the sudden failure of the dam.

20.7.3. Structural failures. About 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

(*a*) *Foundation slide.* (*i.e.* overall stability of the dam). When the foundation of the earth dams are made of soft soils, such as fine silt, soft clay, etc., the entire dam may slide over the foundation. Sometimes, seams of fissured rocks, shales or soft clay, etc. may exits under the foundation, and the dam may slide over some of them, causing its failure. In this type of failure, the top of embankment gets cracked and subsides, the lower slope moves outward forming large mud waves near the heel, as shown in Fig. 20.8.

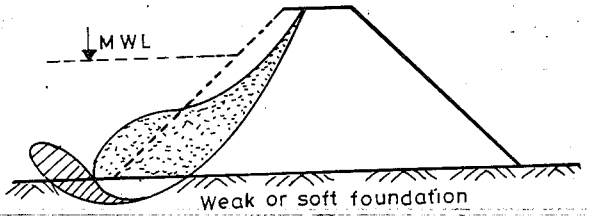


Fig. 20.8. Sliding due to soft or weak foundation.

Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embedded between sands or silts, etc. may reduce the shear strength of the soil, until it becomes incapable of resisting the induced shear stresses, leading to the failure of the dam foundation whithout warning. Loose sand foundations may fail by the liquefaction or flow slides.

(*b*) *Slide in Embankments.* When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure.

The most critical condition of the slide of the u/s slope is the sudden draw-down of the reservoir (Fig. 20.9); and the d/s slope is most likely to slide, when the reservoir is full (Fig. 20.10). The u/s slope failures seldom lead to catastrophic failures, but the d/s slope failures are very serious. These failures, generally occur due to development of excessive unaccounted pore pressures which may reduce the shearing strength of the soils as explained in the previous article. Many embankments may fail during the process of consolidation, at the time of construction or after the construction.

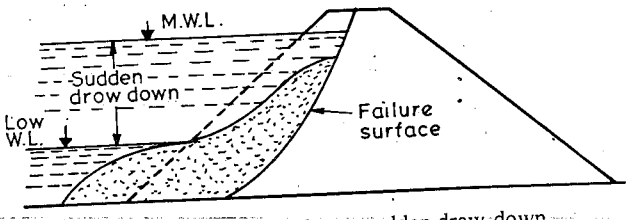


Fig. 20.9. U/S slope slide due to sudden draw-down.

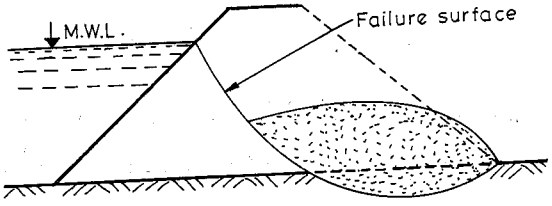


Fig. 20.10. D/S. slope slide during full reservoir condition.

20.8. Design Criteria for Earth Dams

(1) A fill of sufficiently low permeability should be developed out of the available materials, so as to best serve the intended purpose with minimum cost. Borrow pits should be as close to the dam site as possible, so as to reduce the leads.

(2) Sufficient spillway and outlets capacities should be provided so as to avoid the possibility of overtopping during design flood.

(3) Sufficient freeboard must be provided for wind set-up, wave action, frost action and earthquake motions.

(4) The seepage line (*i.e.* phreatic line) should remain well within the downstream face of the dam, so that no sloughing of the face occurs.

(5) There is little harm in seepage through a flood control dam. if the stability of foundations and embankments is not impaired, by piping, sloughing, etc. : but a conservation dam must be as watertight as possible.

(6) There should be no possibility of free flow of water from the upstream to the downstream face.

(7) The upstream face should be properly protected against wave action, and the downstream face against rains and against waves upto tail water. Provisions of horizontal berms at suitable intervals in the d/s face may be thought of, so as to reduce the erosion due to flow of rain water. Ripraps should be provided on the entire u/s slope and also on the d/s slope near the toe and up to slightly above the tail water so as to avoid erosion.

(8) The portion of the dam, downstream of the impervious core, should be properly drained by providing suitable horizontal filter drain, or toe drain, or chimney drain, etc.

(9) The upstream and downstream slopes should be so designed as to be stable under worst conditions of loading. These critical conditions occur for the u/s slope during sudden drawdown of the reservoir, and for the d/s slope during steady seepage under full reservoir.

(10) The u/s and d/s slope should be flat enough, as to provide sufficient base width at the foundation level, such that the maximum shear stress developed remains well below the corresponding maximum shear strength of the soil, so as to provide a suitable factor of safety.

(11) We know that the consolidation of the soil does not take place instantaneously when the compaction is done by external leadings. It takes place slowly as the excess pore water goes out and the load is transferred to the soil grains. In coarse gravels, the void openings are large enough so as to permit rapid escape of confined water and air, and full compaction may occur before the construction is over. But in fine grained impervious soils, the consolidation is slow. It, therefore, becomes necessary in such cases, as to provide an additional height of the fill. After consolidation, the the embankment will be of the desired height. Hence, a suitable allowance in the height of embankment (between 2 to 3% of dam height, determined by laboratory tests) must be made in fine grained soils so as to account for the consolidation that may take place upto years after construction. Dewatering the foundations may sometimes be used to accelerate the process of consolidation.

(12) Since the stability of the embankment and foundation is very critical during construction or even after construction (*i.e.* during the period of consolidation), due to development of excessive pore pressures and consequent reduction in shear strength of soil, the embankment slopes must remain safe under this critical condition also.

All the above criteria must be satisfied and accounted for, in order to obtain a safe design and construction of an earth dam.

20.9. Selecting a Suitable Preliminary Section for an Earth Dam

The preliminary design of an earth dam is done on the basis of existing dams of similar characteristics and the design is finalised by checking the adequacy of the selected section for the worst loading conditions. Empirical rules are frequently used in these designs.

A few recommendations, for selecting suitable values of top width, free board, u/s and d/s slopes, drainage arrangements, etc. are given below for preliminary designs :

Freeboard. Freeboard or minimum freeboard is the vertical distance between the maximum reservoir level and the top of the dam (*i.e.* the crown or crest of dam). The vertical distance between normal pool level or spillway crest and the top of the dam is termed as normal freeboard.

The minimum height of the freeboard for wave action is generally taken to be equal to 1.5 h_w, where h_w is given by the equations (19.11) and (19.12). Most of the hydraulic failures of earth dams have occurred due to overtopping of dams. Hence, the freeboard must be sufficient enough, as to avoid any such possibility of overtopping. Values of freeboard, for various heights, recommended by U.S.B.R. are given in table 20.1.

Table 20.1. U.S.B.R. Recommendations for Freeboard for Earth Dams

Spillway Type	*Height of Dam*	*Minimum freeboard Over MWL*
Uncontrolled (*i.e.* Free) Spillway	Any height	Between 2 m to 3 m
Controlled spillway	Height less than 60 m	2.5 m above top of gates
Controlled spillway	Height more than 60 m	3 m above top of gates

An additional freeboard upto 1.5 m should be provided for dams situated in areas of low temperatures for frost action.

Width. The top width of large earthen dams should be sufficient to keep the seepage line well within the dam, when reservoir is full. It should also be sufficient to withstand earthquake shocks and wave action. For small dams, this top width is generally governed by minimum roadway width requirements.

The top width (A) of the earth dam can be selected as per the following recommendations :

$$A = \frac{H}{5} + 3 \text{ for very low dams} \quad ...(20.11)$$

$$A = 0.55\sqrt{H} + 0.2H \text{ for dams lower than 30 m} \quad ...(20.12)$$

$$A = 1.65\,(H + 1.5)^{1/3} \text{ for dams higher than 30 m} \quad ...(20.13)$$

where H is the height of the dam.

Upstream and Downstream slopes. The side slopes depend upon various factors such as the type and nature of the dam, and foundation materials, height of dam, etc. etc. The recommended values of side slopes as given by Terzaghi are tabulated in Table 20.2.

Table 20.2. Terzaghi's Side Slopes for Earth Dams

Type of Material	*U/S slope (H : V)*	*D/S slope (H : V)*
Homogeneous well graded	2.5 : 1	2 : 1
Homogeneous course silt	3 : 1	2.5 : 1
Homogeneous silty clay		
(*i*) Height less than 15 m	2.5 : 1	2 : 1
(*ii*) Height more than 15 m	3 : 1	2.5 : 1
Sand or Sand and gravel with a central clay core	3 : 1	2.5 : 1
Sand or Sand and gravel with R.C. diaphragm	2.5 : 1	2 : 1

The various dimensions of low earth dams for their preliminary sections, may sometimes be selected from the recommendations of Strange, as given in Table 20.3.

Table 20.3. Preliminary Dimensions of Low Earth Dams (Strange's recommendations)

Height of dam in metres	*Maximum freeboard in metres*	*Top width (A) in metres*	*U/S slope (H : V)*	*D/S slope (H : V)*
Up to 4.5	1.2 to 1.5	1.85	2 : 1	1.5 : 1
4.5 to 7.5	1.5 to 1.8	1.85	2.5 : 1	1.75 : 1
7.5 to 15	1.85	2.5	3 : 1	2 : 1
15 to 22.5	2.1	3.0	3 : 1	2 : 1

SEEPAGE ANALYSIS

Seepage occurs through the body of all earthen dams and also through their pervious foundations. The amount of seepage has to be controlled in all conservation dams and the effects of seepage (*i.e.* position of phreatic line) has to be controlled for all dams, in order to avoid their failures.

The seepage through a pervious soil material, for two dimensional flow, is given by Laplacian equation

$$\frac{\partial^2\phi}{\partial x^2}+\frac{\partial^2\phi}{\partial y^2}=0 \qquad \text{...(20.14)}$$

where $\phi = K \cdot h$ = Velocity potential
K = Permeability of the soil
h = Head causing flow.

The above equation is based on the following assumptions :

(*i*) Water is incompressible.

(*ii*) The soil is incompressible and porous. The size of the pore space do not change with time regardless of water pressure.

(*iii*) The quantity of water entering the soil in any given time is the same as the quantity flowing out of the soil.

(*iv*) Darcy's law is valid for the given soils.

(*v*) The hydraulic boundary conditions at the entry and exit are known.

A graphical solution of the above equation, (*i.e.* Eq. 20.11) suggests that the flow through the soil, following the above assumptions, can be represented by a *flow-net* ; which consists of two sets of curves, known as '*Equipotential lines*' (*i.e.* lines of equal energy) and '*stream lines*' (*i.e.* flow lines), mutually perpendicular to each other, as shown in Fig. 20.11.

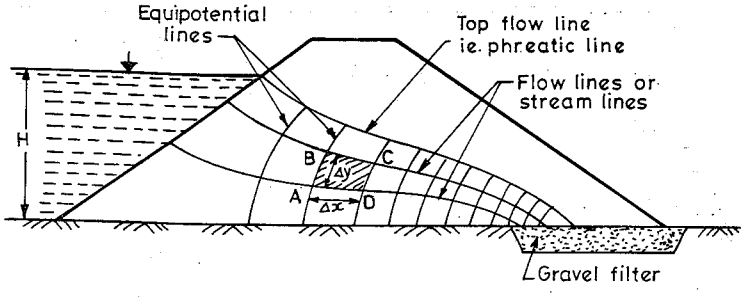


Fig. 20.11. Flow Net.

20.10. Seepage Discharge Through the Isotropic Soils

The amount of seepage can be easily computed from the flownet. Let us assume that the soil is isotropic, *i.e.* its permeability is constant in all directions, or $K_H = K_V$. (*i.e.* horizontal permeability is equal to the vertical permeability). The dam section is first of all plotted to a given scale (Same scale for horizontal and vertical directions). The flow net is drawn by free hand sketching the making suitable adjustments and corrections until the flow lines and equipotential lines intersect at right angles. It is convenient to draw only a limited number of flow lines and equipotential lines, such

that the rate of flow between each pair of flow lines (called flow channels) is equal and the energy drop between any two successive potential lines is the same. The distance between the flow lines is made equal to the distance between the potential lines, thus forming a series of squares. Where the flow lines are curved, the squares formed will be distorted, but they will be more perfect as the number of lines is increased.

The seepage rate (q) can be computed from the flow net, using Darcy's Law. Applying the principle of continuity between each pair of flow lines, it is evident that the velocity must vary inversely with the spacing. Assuming the dam cross-section of Fig. 20.11 to have a unit width, we have :

The flow through the square $ABCD$ (called field) or through the flow channel containing this square

$$= \Delta q = K \cdot iA$$

$$= K\left(\frac{\Delta H}{\Delta x}\right)(\Delta y \times 1)$$

where ΔH is the energy drop between the two equi-potential lines bounding the square $ABCD$ and Δx and Δy are defined in Fig. 20.11.

$$\therefore \qquad \Delta q = K\left(\frac{\Delta H}{\Delta x}\right)\Delta y$$

But $$\Delta H = \frac{\text{Total drop, } i.e. \text{ total head causing flow}}{\text{Number of increments into which the total drop is equally divided}}$$

or $$\Delta H = \frac{H}{N_d}$$

where N_d = Total number of drops in the complete flow-net.

$$\Delta q = K\frac{H}{N_d}\left(\frac{\Delta y}{\Delta x}\right)$$

$$= \frac{K \cdot H}{N_d} \text{ (since } \Delta y = \Delta x)$$

The total flow through all the channels, *i.e.* the total flow through the unit width of the dam

$$= q = \Sigma \cdot \Delta q$$

$$= \left(\frac{K \cdot H}{N_d}\right) \times \text{number of flow channels}$$

or $$\boxed{q = \frac{K \cdot H}{N_d} N_f} \qquad \text{...(20.15)}$$

This is the required expression, representing discharge passing through a flow net and is applicable only to isotropic soils (*i.e.* soils for which $K_H = K_V$).

20.11. Seepage Discharge for Non-isotropic Soils

If the permeability of the soil is different in the horizontal direction than that in the vertical direction ; the flow net is drawn in the same manner as was explained earlier

for isotropic soils, with the only difference that the dam section shall be drawn to the same vertical scale *but to a transformed horizontal scale.* All horizontal dimensions shall be reduced by multiplying them by a factor equal to $\sqrt{\frac{K_V}{K_H}}$. Flow net and squares will be drawn in the same manner, and number of flow channesl (N_f) and number of drops (N_d) shall be counted. The discharge can then be computed by the equation

$$q = \sqrt{K_H \cdot K_V}\ \frac{H \cdot N_f}{N_d} \qquad ...(20.16)$$

20.12. Line of Seepage or Phreatic Line in Earth Dams

Line of seepage or phreatic line or saturation line is defined as the line within the dam section below which there are positive hydrostatic pressures in the dam. The hydrostatic pressure on the phreatic line is equal to the atmospheric pressure and hence, equal to zero. Above the phreatic line, there is a zone of capillary saturation called capillary fringe, in which the hydrostatic pressures are negative. The appreciable flow through the dam body below the phreatic line, reduces the effective weight of this soil, and thus reduces the shear strength of the soil due to pore pressure. But on the other hand, the insignificant flow through the capillary fringe, leads to greater shear strength, because the capillary tension in water leads to increased intergranular pressure. The effects of the capillary fringe are thus on a slightly safer side and hence neglected.

It is, therefore, absolutely essential to determine the position of the phreatic line, as its position will enable us to determine the following things :

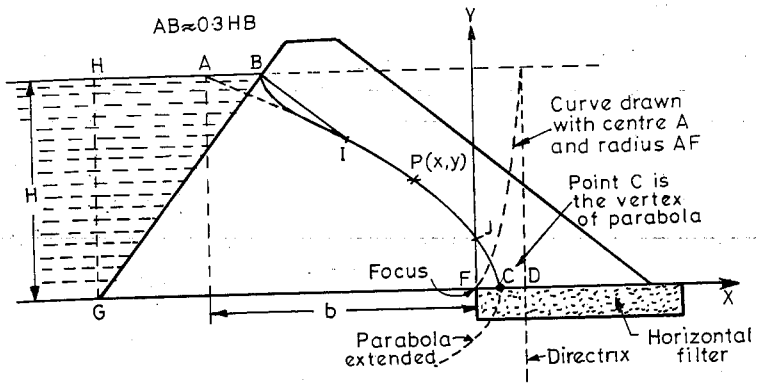
(*i*) It gives us a divide line between the dry (or moist) and submerged soil. The soil above the seepage line will be taken as dry and the soil below the seepage line shall be taken as submerged for computation of shear strength of soil.

(*ii*) It represents the top streamline and hence, helps us in drawing the flow net.

(*iii*) The seepage line determination, helps us to ensure that it does not cut the downstream face of the dam. This is extremely necessary for preventing softening or sloughing of the dam.

20.12.1. Determination of Phreatic Line when the Dam section is Homogeneous and Provided with a Horizontal Filter. It has been found by experiments that the seepage line is pushed down by the filter and it is very nearly parabolic except near its junction with the u/s face. Since the u/s face of the dam (*i.e. GB* in Fig. 20.12) becomes an equipotential line when fully covered with water (water shall rise up to the same piezometric head at every point of this line), the seepage line shall be perpendicular to this face near its junction point *B*. Let a base parabola with focus at *F* is drawn and produced so as to intersect the water surface at a point *A*. Cassagrande has shown that for dams with reasonably flat upstream slopes, $AB \approx 0.3\,HB$, where *H* is the projection of the point *G* (*i.e.* the end point of u/s slope) on the water surface. Knowing the point *A*, the base parabola *AIJC* can be drawn with its focus at *F*. It can then be corrected for the curve *BI* such that *BI* is perpendicular to *GB*, thus, *BIJC* will finally represent the seepage line.

Equation of the base parabola. The equation of the parabola can be determined from the basic property of the parabola *viz., the distance of any point P (x, y) on the parabola from its focus is the same as the distance of that point P (x, y) from a line*



Point C is the vertex of the parabola
Fig. 20.12

called directrix. Taking the focus (F) as the origin, the equation of the parabola can be written as

$$\sqrt{x^2+y^2} = x + FD$$

where the vertical line through D is the directrix.

ED is the distance of the focus from the directrix, called focal distance and is generally represented by S.

The equation of the parabola, then becomes

$$\sqrt{x^2+y^2} = x + S \qquad ...(20.17)$$

If the horizontal distance between the already determined point A and the focus (F) is taken as say b, then (b, H) represents the coordinates of the point A on the parabola. Substituting in equation (20.17), we get

$$\sqrt{b^2+H^2} = b + S$$

or
$$S = \sqrt{b^2+H^2} - b \qquad ...(20.18)$$

S can be calculated from this equation.

The focal distance S can also be measured by drawing an arc FH with centre A and radius equal to AF so as to intersect the Horizontal water surface AB produced at H. The vertical line HD through H will then represent the directrix. FD will be equal to S. The centre point (C) of FD will be the vertex of the parabola.

Also from equation (20.17) : when $x = 0$, $y_{(0)} = S$. Hence, the vertical ordinate FJ at F will be equal to S. Hence, S is many a times, represented by $y_{(0)}$. Knowing the points A, C, J and working out a few more points from the equation (20.17), the parabola can be easily drawn and corrected for the curve BI, so as to get the seepage line $BIJC$.

The discharge can also be calculated easily from the equation of the seepage line, without taking the trouble of drawing a flow net, as explained below :

Consider a unit width of the dam. Let q be the seepage discharge per unit width of the dam. Then, according to Darcy's Law, $q = KiA$. When steady conditions have

reached, the discharge crossing any vertical plane across the dam section will be the same. Hence, the values of i and A can be taken for any point on the seepage line

$$\therefore \qquad i = \frac{dy}{dx}$$

$$A = y \times 1 \text{ (}i.e.\text{ saturated depth} \times \text{width)}$$

$$\therefore \qquad q = K \frac{dy}{dx} \cdot y \qquad \text{...(20.19)}$$

But the equation of the parabola is

$$\sqrt{x^2 + y^2} = x + S \qquad \text{...(20.17)}$$

or $$x^2 + y^2 = (x + S)^2$$

or $$y^2 = (x + S)^2 - x^2$$

or $$y = \sqrt{S^2 + 2xS}.$$

Equation (20.19) becomes

$$q = K\left[\frac{1}{2} \cdot (S^2 + 2xS)^{\frac{1}{2}-1} \cdot 2S\right] \cdot \left[\sqrt{S^2 + 2xS}\right]$$

$$= K \cdot S \cdot (S^2 + 2xS)^{-\frac{1}{2}} \sqrt{S^2 + 2xS}$$

$$= K \cdot S \cdot \frac{\sqrt{S^2 + 2xS}}{\sqrt{S^2 + 2xS}} = K \cdot S.$$

or $$\boxed{q = KS} \qquad \text{...(20.20)}$$

The coefficient of permeability K and the focal distance (S) are known ; the discharge q can be easily computed. This is an important equation. Strictly speaking, this equation is applicable only to dams with horizontal drainage but is used for other type of section also and gives quite close values of discharge.

20.12.2. Determination of Phreatic Line when the Dam Section is Homogeneous (without Filter). The phreatic line can be determined on the same principles as was done for dam with a filter case. The focus (F) of the parabola, in this case, will be the lowest point of the downstream slope as shown in Fig. 20.13. The base parabola $BIJC$ will cut the downstream slope at J and extend beyond the dam toe up to the point C (*i.e.* the vertex of the parabola).

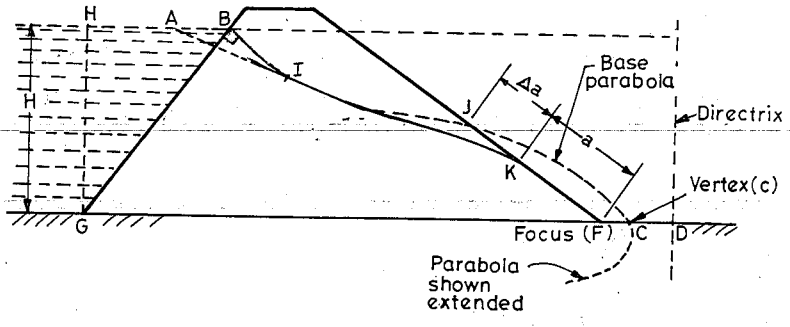


Fig. 20.13

The seepage line will, however, emerge out at K, meeting the downstream face tangentially there. *The portion KF is known as discharge face and always remains saturated.* The correction JK (say Δa) by which the parabola is to be shifted downward can be determined as follows :

(A) Graphical general solution. Cassagrande has given a general solution to evaluate Δa for various inclinations of discharge face. Let a be the angle which the discharge face makes with the horizontal. The various values of $\frac{\Delta a}{a+\Delta a}$ have been given by Cassagrande, as shown in table 20.4.

Table 20.4

α *in degrees*	$\frac{\Delta a}{a+\Delta a}$	*Remarks*
30°	0.36	*Note.* Intermediate values can be interpolated, or read out from a graph between α and $\frac{\Delta a}{a+\Delta a}$ plotted with the values given here.
60°	0.32	
90°	0.26	
120°	0.18	
135°	0.14	
150°	0.10	
180°	0.0	

$(a+\Delta a)$ is the distance FJ (*i.e.* the distance of the focus from the point where the parabola cuts the d/s face) and is known. Δa can then be evaluated. a and Δa can be connected by a general equation.

$$\Delta a = (a+\Delta a)\left[\frac{180^\circ - \alpha}{400^\circ}\right] \qquad ...(20.21)$$

The value of α will be equal to 180° for a horizontal filter case and may be equal to or more than 90° in case a rock toe is provided at the downstream end, as shown in Fig. 20.14 (*a*). α will be less then 90° when no drainage is provided.

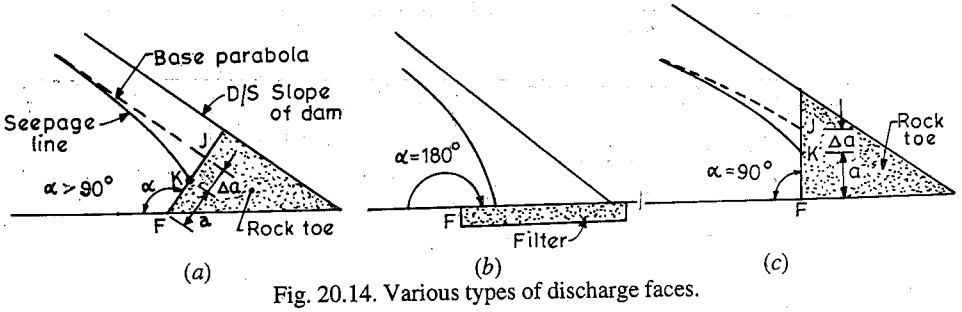


Fig. 20.14. Various types of discharge faces.

(B) Analytical solutions for determining the position of point k, *i.e.* the point at which the seepage line intersects the d/s slope.

Case (*a*) when $\alpha < 30^\circ$

Schaffernak and Van Iterson have derived an equation for determining the value of 'a' (and thus fixing the position of point K) in terms of H, b' and α. Their final equation is

$$a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b'^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}} \qquad ...(20.22)$$

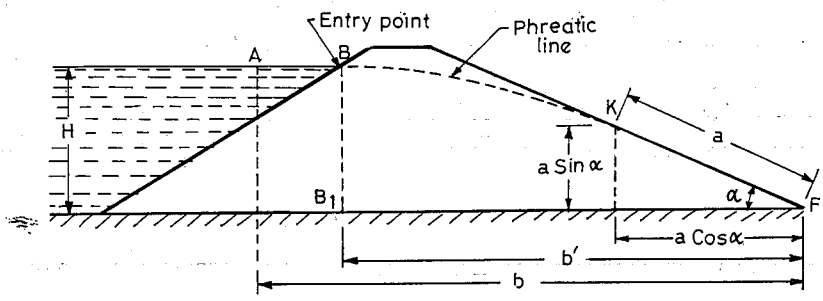


Fig. 20.15

The above equation has been obtained on the assumption that the hydraulic gradient (i) is equal to the slope of the phreatic line. This assumption is nearly true so long as the downstream slope is sufficiently flat (*i.e.* $\alpha < 30°$).

This equation can be easily derived on the basis of the above assumption as follows:

$$i = \text{Hydraulic gradient} = \frac{dy}{dx}$$

$$A = y \cdot 1$$

$$q = KiA = K \cdot \left(\frac{dy}{dx}\right) \cdot (y \cdot 1)$$

But $\frac{dy}{dx} = \tan \alpha$

and $y = a \sin \alpha$

$$\therefore \quad q = K \cdot \tan \alpha \cdot a \cdot \sin \alpha$$

$$= K \cdot a \cdot \sin \alpha \cdot \tan \alpha$$

or $$K \cdot \frac{dy}{dx}(y) = K \cdot a \cdot \sin \alpha \tan a$$

or $$\frac{dy}{dx}(y) = a \sin \alpha \tan \alpha$$

or $$dy \cdot y = a \sin \alpha \tan \alpha \, dx$$

or $$y \cdot dy = a \sin \alpha \tan \alpha \, dx.$$

Integrating both sides between the limits

$$x = a \cos \alpha \quad \text{to} \quad x = b'$$

and $$y = a \sin \alpha \quad \text{to} \quad y = H,$$

we get $$\int_{y = a \sin \alpha}^{y = H} y \cdot dy = a \sin \alpha \cdot \tan \alpha \int_{x = a \cos \alpha}^{x = b'} dx$$

or $$\left| \frac{y^2}{2} \right|_{a \sin \alpha}^{H} = a \sin \alpha \tan \alpha \, \Big| \, x \, \Big|_{a \cos \alpha}^{b'}$$

or $$\frac{H^2 - a^2 \sin^2 \alpha}{2} = a \sin \alpha \tan \alpha \left[b' - a \cos \alpha \right]$$

or $$\frac{H^2}{2} - \frac{a^2}{2} \sin^2 \alpha = ab' \sin \alpha \tan \alpha - a^2 \cdot \sin \alpha \cos \alpha \cdot \tan \alpha$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = ab' \sin\alpha \tan\alpha - a^2 \cdot \sin^2\alpha$$

or $$\frac{H^2}{2} = ab' \sin\alpha \tan\alpha - \frac{a^2}{2}\sin^2\alpha$$

or $$\frac{a^2}{2} \cdot \sin^2\alpha - ab' \sin\alpha \tan\alpha + \frac{H^2}{2} = 0$$

or $$a^2 \sin^2\alpha - 2ab' \sin\alpha \tan\alpha + H^2 = 0$$

or $$a^2 - \frac{2ab'}{\cos\alpha} + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{\frac{2b'}{\cos\alpha} \pm \sqrt{\left(\frac{2b'}{\cos\alpha}\right)^2 - \frac{4H^2}{\sin^2\alpha}}}{2}$$

Ignoring unfeasible +ve sign, we have

$$\boxed{a = \frac{b'}{\cos\alpha} - \sqrt{\frac{b^2}{\cos^2\alpha} - \frac{H^2}{\sin^2\alpha}}} \quad ...(20.22)$$

This is the required equation.

Case (*b*). When α lies between 30° and 60°

Cassagrande has derived an equation for determining the value of '*a*' in terms of *b*, *H* and α. His final equation is

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2 \cot^2\alpha} \quad ...(20.23)$$

where *b* is defined in Fig. 20.16,

H is the head causing flow and α is the angle which the d/s face makes with the horizontal (clockwise) as defined earlier.

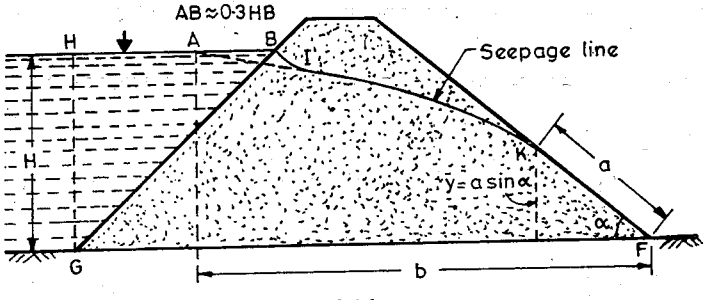


Fig. 20.16

Equation (20.23) gives satisfactory results for values of α less than 30°, but for steeper slopes, the deviations from the correct values become quite high and for such cases, Cassagrande has suggested the use of sin α in place of tan α.

In other words, the hydraulic gradient (*i*) is given by $\frac{dy}{ds}$ (*i.e.* sin α) of the seepage line and not by $\frac{dy}{dx}$ (*i.e.* tan α) as was taken in the previous case.

Therefore, $q = K \cdot i \cdot A$

$$\left(\frac{dy}{ds}\right) y \cdot 1 = K(\sin\alpha)(a\sin\alpha) \qquad \because \frac{dy}{ds} = \sin\alpha$$

1065

$$K \cdot a \cdot \sin^2\alpha$$

$$y = a\sin^2\alpha \cdot ds.$$

ween the limits

$y = a\sin\alpha, \quad s = a$

$y = H \quad s = S_0$

where S_0 is the total length of parabola from the point A to the point F.

or $$\int_{y=a\sin\alpha}^{y=H} y \cdot dy = a \cdot \sin^2\alpha \int_{s=a}^{s=S_0} ds$$

or $$\left|\frac{y^2}{2}\right|_{a\sin\alpha}^{H} = a\sin^2\alpha \cdot |s|_{s=a}^{s=S_0}$$

or $$\frac{H^2 - a^2\sin^2\alpha}{2} = a\sin^2\alpha\,[S_0 - a]$$

or $$\frac{H^2}{2} - \frac{a^2}{2}\sin^2\alpha = a\sin^2\alpha \cdot S_0 - a^2\sin^2\alpha$$

or $$\frac{a^2}{2}\sin^2\alpha - S_0\sin^2\alpha \cdot a + \frac{H^2}{2} = 0$$

or $$a^2 - 2S_0 \cdot a + \frac{H^2}{\sin^2\alpha} = 0$$

or $$a = \frac{2 \cdot S_0 \pm \sqrt{4 \cdot S_0^2 - 4 \cdot \frac{H^2}{\sin^2\alpha}}}{2}$$

Ignoring +ve sign, we get

$$a \doteq S_0 - \sqrt{S_0^2 - \frac{H^2}{\sin^2\alpha}} \qquad \text{...(20.24)}$$

The total length of the parabola S_0 can be approximately taken to be equal to $\sqrt{b^2 + H^2}$, then

$$S_0 = \sqrt{b^2 + H^2}$$

Substituting, we get

$$a = \sqrt{b^2 + H^2} - \sqrt{b^2 + H^2 - \frac{H^2}{\sin^2\alpha}}$$

or $$a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\left(\frac{1}{\sin^2\alpha} - 1\right)}$$

$$\boxed{a = \sqrt{b^2 + H^2} - \sqrt{b^2 - H^2\cot^2\alpha}}$$ which is the required eqn. (20.23).

Example 20.1. *An earth dam made of a homogeneous material has the following data :*

Coefficient of permeability of dam material	$= 5 \times 10^{-4}$ *cm/sec.*
Level of top of dam	$= 200.0$ *m.*
Level of deepest river bed	$= 178.0$ *m*
H.F.L. of reservoir	$= 197.5$ *m.*
Width of the top of dam	$= 4.5$ *m.*
Upstream slope	$= 3 : 1.$
Downstream slope	$= 2 : 1.$

Determine the phreatic line for this dam section and the discharge passing through the dam.

Solution. Taking the focus (F) at the d/s toe of the dam as the origin, the equation of the base parabola is given by $\sqrt{x^2 + y^2} = x + S$

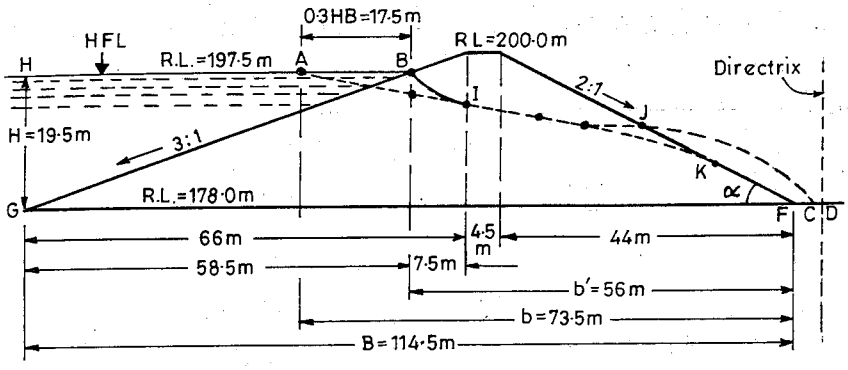


Fig. 20.17

where S is the distance of the point (x, y) from the directrix, called focal distance. Select the point A, in such a way that

$$AB \approx 0.3\, HB$$

or $$AB \approx 0.3 \times 58.5 = 17.5 \text{ m}.$$

Plot the point A at a distance 17.5 m from B (Fig. 20.17). This is the start-point of the parabola. Now the coordinates of the point A w.r. to F as origin are (73.5 m, 19.5 m). Substituting these ordinates in the equation of the base parabola, we get

$$\sqrt{(73.5)^2 + (19.5)^2} = 73.5 + S.$$

or $$\mathbf{S = 2.54 \text{ m}.}$$

The vertex (C) of the base parabola shall be situated at a distance equal to $\frac{S}{2}$ or 1.27 m from F, beyond the d/s toe of the dam, as shown in Fig. 20.17.

A few more coordinates of the base parabola at known distances (x) are worked out in table 20.4 using,

$$x^2 + y^2 = x^2 + S^2 + 2xS$$

or $$y = \sqrt{S^2 + 2xS}$$

Table 20.4

x	$y^2 = 2xS + S^2 = 5.08x + 6.45$	$y = \sqrt{S^2 + 2xS}$
0	6.45	2.54
10	57.25	7.68
20	108.05	10.39
30	158.85	12.61
40	209.65	14.48
44	229.97	15.16
48.5	252.83	15.92
56	290.93	17.04
60	311.25	17.64
70	362.05	19.02
73.5	380.00	19.50

The base parabola with all these ordinates, is then drawn.

Now, this parabola has to be corrected at entry and exit as explained earlier. At entry, the phreatic line is started from the point B in such a way that it becomes at right angles to the u/s face GB of the dam. A reverse curvature BI is, therefore, given as shown in Fig. 20.17.

At exit, the point K at which the phreatic line intersects the d/s face can be easily obtained by using the Eq. (20.21) as :

$$\Delta a = (a + \Delta a)\left(\frac{180 - \alpha}{400}\right)$$

where $\tan\alpha = \frac{1}{2}$; or $\alpha = 26° \cdot 54$

$(a + \Delta a)$ = Distance FJ, *i.e.* the distance of the focus from the point at which the base parabola intersects the d/s face, and is measured from Fig. 20.17 ≈ 25.6 m.

$$\therefore \quad \Delta a = 25.6\left[\frac{180 - 26.54}{400}\right] = 25.6 \times \frac{153.46}{400} = 9.84 \text{ m ; say } \mathbf{9.8 \text{ m.}}$$

$$a = 25.6 - 98 = 15.8 \text{ m.}$$

Knowing 'a', the point K is plotted and the phreatic line BIK is completed.

Discharge through the dam section can be obtained from the equation (20.20) as :

$$q = K \cdot S, \qquad \text{where } K = 5 \times 10^{-4} \text{ cm/sec} = 5 \times 10^{-6} \text{ m/sec.}$$
$$S = 2.54 \text{ m}$$

$$\therefore \quad q = 5 \times 10^{-6} \times 2.54 \text{ m}^3/\text{m run/sec.}$$
$$= 12.7 \times 10^{-6} \text{ cumes/m length of dam}$$

Hence, $q = 12.7 \times 10^{-6}$ cumecs/m length of dam. **Ans.**

Note. The value of 'a' can also be determined from Eq. (20.22), if $(a + \Delta a)$ is not to be measured, as the dam section is not to be plotted to scale.

In that case $$a = \frac{b'}{\cos a} - \sqrt{\left(\frac{b'}{\cos\alpha}\right)^2 - \left(\frac{H}{\sin\alpha}\right)^2}$$

where $b' = 56$ m

$H = 19.5$ m

$\sin 26.56° = 0.447$

$\cos 26.54° = 0.894.$

$$\therefore \quad a = \frac{56}{0.894} - \sqrt{\left(\frac{56}{0.894}\right)^2 - \left(\frac{19.5}{0.447}\right)^2}$$

$$= 62.6 - \sqrt{3{,}920 - 1{,}900} = 62.6 - 44.9 = \mathbf{17.7\ m.}$$

Hence, $a = 17.7$ m which is slightly above the value obtained from Eq. (20.21) and is thus on a safer side.

Example 20.2. *A flow net is plotted for a homogeneous earthen dam of height 22 m and freeboard 2.0 m. The results obtained are,*

Number of potential drops = 10

Number of flow channels = 4.

The dam has a horizontal filter of 30 m length at the downstream end and the coefficient of permeability of the dam material is 5×10^{-4} *cm/sec. Calculate the discharge per m run of the dam.*

Solution. The discharge through a dam section is approximately given by the Eq. (20.15) as :

$$q = K.H.\frac{N_f}{N_d}$$

where $K = 5 \times 10^{-4}$ cm/sec $= 5 \times 10^{-6}$ m/sec.

$H = 22 - 2 = 20$ m.

$N_f = 4$

$N_d = 10$

$$\therefore \quad q = 5 \times 10^{-6} \times \frac{20 \times 4}{10}$$

$$= \mathbf{4.0 \times 10^{-6}\ cumecs/m\ run\ of\ dam.}\ \textbf{Ans.}$$

Example 20.3. *For the dam section given in example 20.1, draw the seepage line if a horizontal filter of length equal to 25 m is provided inward from the downstream toe of the dam.*

Solution. The dam section is plotted from the dimensions given in example 20.1. The horizontal filter is also provided. Now, the left end of the filter will act as a focus and is designated as F and is taken as origin.

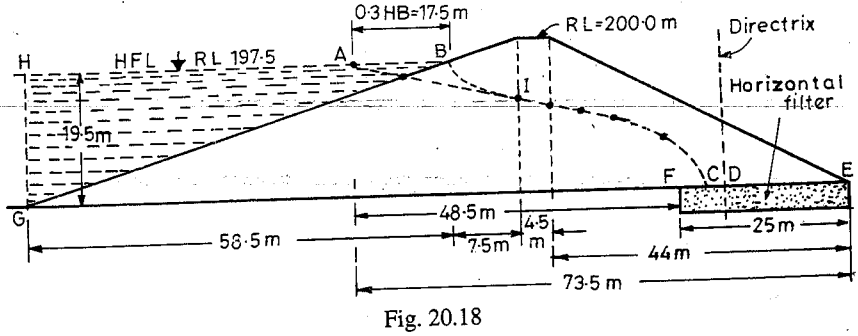


Fig. 20.18

The coordinates of any point (x, y) on a parabola of equation

$$\sqrt{x^2+y^2} = x+S$$

will now be for the point A.

The point A is determined as in example 20.1. Its coordinates w.r. to F as origin are (48.5 m, 19.5 m)

$$\therefore \quad \sqrt{(48.5)^2+(19.5)^2} = 48.5+S$$

or $$\sqrt{2,352+380} = 48.5+S$$

or $$\mathbf{S = 3.77\ m.}$$

The vertex (C) of the parabola shall be situated at a distance equal to $\frac{S}{2}$, *i.e.* 1.83 m beyond the point F and directrix shall be at a distance 3.77 m from F.

At point F, $x = 0$

$\therefore$ $y = S = 3.77$ m.

A few more coordinates of the base parabola are worked out in table 20.5, using

$$x^2+y^2 = x^2+S^2+2xS$$

or $$y = \sqrt{2xS+S^2}$$

Table 20.5

x	$y^2 = (2xS+S^2) = 7.54x + 14.21$	$y = \sqrt{2xS+S^2}$
0	14.21	3.77
10	89.61	9.47
15	127.31	11.27
19	157.47	12.54
23.5	191.40	13.85
31	247.95	15.75
40	315.81	17.78
48.5	380.00	19.5

The base parabola is drawn and correction at the entry point for the curve BI is made by free hand sketching, in such a way that the phreatic line becomes $\perp$ to u/s face GB of the dam. The exit point should also be corrected such that the phreatic line meets $\perp$ to base line FCD. The final phreatic line BIC is thus drawn as shown in Fig. 20.18.

20.12.3. Phreatic Line for a Zoned Section. In case of a zoned section having a central impervious core, such as shown in Fig. 20.19 ; the effects of the outer zone can be neglected altogether.

The focus of the base parabola will, therefore, be located at the d/s toe of the core, as shown in Fig. 20.19. The phreatic line can then be drawn as usual with free hand corrections required at suitable places.

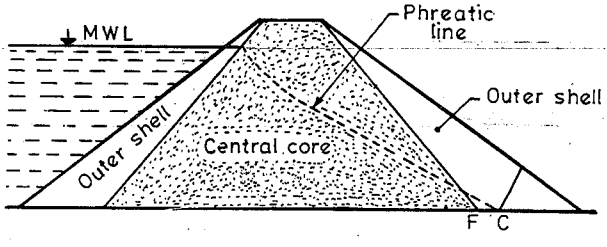


Fig. 20.19

20.13. Stability of Earthern Slopes

An earth embankment usually fails, because of the sliding of a large soil mass along a curved surface. It has been established by actual investigation of slides of railway embankments in Sweden, that the surface of slip is usually close to cylindrical, *i.e.* an arc of a circle in cross-section. The method which is described here and is generally used for examining the stability of slopes of an earthen embankment is called the *Swedish Slip Circle Method* or *The Slices Method.* The method thus assumes the condition of plane strain with failure along a cylindrical arc.

The location of the centre of the possible failure arc is assumed. The earth mass is divided into a number of vertical segments called slices. These verticals are usually equally spaced, though it is not necessary to do so. Depending upon the accuracy desired, six to twelve slices are generally sufficient.

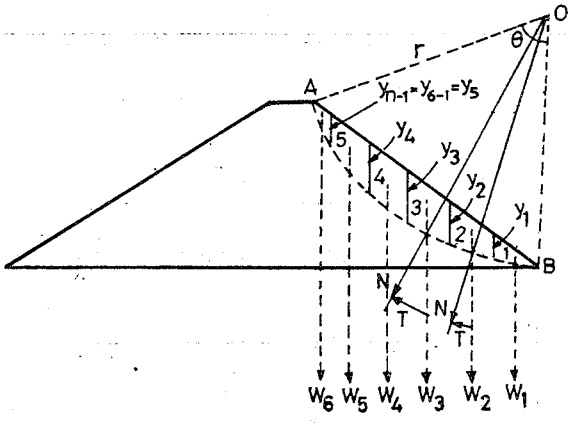


Fig. 20.20

Let O be the centre and r be the radius of the possible slip surface as shown in Fig. 20.20. Let the total arc AB be divided into slices of equal width say b metres each. The width of the last slice will be something different say let it be $m \cdot b$ metres.

Let these slices be numbered as 1, 2, 3, 4, ... and let the weight of these slices be $W_1, W_2, W_3, W_4 \ldots$

The forces between these slices are neglected and each slice is assumed to act independently as a vertical column of soil of unit thickness ($\perp$ to paper) and width b. The weight W of each slice is assumed to act at its centre. The weight W of each slice can be resolved into two components ; say a normal component (N) and a tangential component (T) such that

$$N = W \cos \alpha$$
$$T = W \sin \alpha$$

where α is the angle which the slope makes with the horizontal.

The normal component (N), will pass through the centre of rotation (O) and hence does not create any moment on the slice. However, the tangential component (T) causes a disturbing moment equal to ($T \cdot r$), where r is the radius of the slip circle. The tangential components of a few slices may create resisting moments; in that case T is considered as negative.

The total disturbing moment (M_d) will be equal to the algebraic sum of all the tangential moments, *i.e.*

$$M_d = \Sigma T \cdot r = r \cdot \Sigma T$$

where $\Sigma T = (T_1 + T_2 + T_3 + \ldots)$

The resisting moment is supplied by the development of shearing resistance of the soil along the arcual surface AB. The magnitude of shear strength developed in each slice will depend upon the normal component (N) of that slice. Its magnitude will be

$$= c \cdot \Delta L + N \tan \phi$$

where c is the unit cohesion, ΔL is the curved length of the slice and ϕ is the angle of internal friction of soil.

This shear resistance is acting at a distance r from O and will provide a resisting moment

$$= r\,[c \cdot \Delta L + N \tan \phi]$$

The total resisting moment over the entire arc AB

$$M_r = r\,[\Sigma C \cdot \Delta L + \Sigma N \cdot \tan \phi]$$
$$= r\,[c \Sigma\, \Delta L + (\Sigma N) \tan \phi]$$
$$= r\,[c \cdot \widehat{AB} + (\Sigma N) \tan \phi]$$

where ΣN is the sum of all the normal components.

$$= N_1 + N_2 + N_3 + \ldots$$

Length $\widehat{AB}$ of slip circle

$$= \widehat{AB} = \left[\frac{2\pi \cdot r}{360^\circ}\right] \times \theta$$

where θ is the angle in degrees, formed by the arc AB at centre O.

Hence, the factor of safety against sliding

$$= \text{F.S.} = \frac{\text{Resisting moment}}{\text{Disturbing moment}} = \frac{M_r}{M_d}$$

or
$$= \frac{r\,[c \cdot \widehat{AB} + (\tan \phi)\, \Sigma N]}{r \cdot \Sigma T}$$

or
$$\text{F.S.} = \frac{[c \cdot \widehat{AB} + (\tan \phi)\, \Sigma N]}{(\Sigma T)} \qquad \ldots(20.24)$$

Eq. (20.24) can be worked out by working out ΣT and ΣN separately. This evaluation of ΣN and ΣT can be simplified as explained below :

If $y_1, y_2, y_3, \ldots$ are the vertical extreme ordinates (boundary ordinates) of the slices 1, 2, 3... respectively, then the respective weights can be written as

$$W_1 = \left(\frac{0 + y_1}{2}\right) \cdot b \cdot \gamma \cdot 1$$

where γ is the unit weight of soil and unit width of the slice is considered.

$$W_2 = \left(\frac{y_1 + y_2}{2}\right) b \cdot \gamma$$

$$W_3 = \left(\frac{y_2 + y_3}{2}\right) b \cdot \gamma$$

$$W_4 = \left(\frac{y_3 + y_4}{2}\right) b \cdot \gamma$$

.....

.....

$$W_n = \left(\frac{y_{n-1} + 0}{2}\right) m \cdot b \cdot \gamma$$

where n is the total number of slices.

$$\therefore \quad \Sigma W = W_1 + W_2 + W_3 + \ldots + W_n$$

$$= \left[y_1 b + y_2 b + y_3 b + \ldots \frac{y_{n-1}}{2} b + \frac{y_{n-1}}{2} m \cdot b \right] \gamma$$

$$= \left[y_1 + y_2 + y_3 + \ldots y_{n-1} \left(\frac{1+m}{2} \right) \right] \gamma \cdot b$$

Now $\Sigma N = N_1 + N_2 + N_3 + \ldots$

$= W_1 \cos\alpha + W_2 \cos\alpha + W_3 \cos\alpha + \ldots$

$\therefore \quad \Sigma N = \cos\alpha \, [W_1 + W_2 + W_3 + \ldots]$

or $\Sigma N = \cos\alpha \, (\Sigma W)$.

Similarly $\Sigma T = \sin\alpha \, (\Sigma W)$ if all T's are +ve.

ΣW can be evaluated by adding all the vertical boundary ordinates of all the slices. (The last ordinate should be multiplied by $\frac{m+1}{2}$ before adding) and then multiplying this summation (say Σy) by $b \cdot \gamma$.

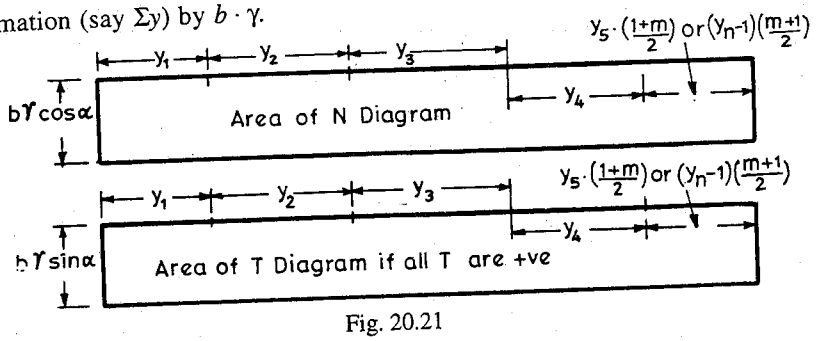


Fig. 20.21

The area of N diagram will represent ΣN and that of T diagram will represent ΣT.

As a general case, the value of ΣN and ΣT can be worked out in a tabular form as shown below in table 20.6.

Table 20.6

Slice number	*Wt. of each slice*	$N = W\cos\alpha$	$T = W\sin\alpha$	$c \cdot \Delta L$
1	W_1	N_1	T_1	
2	W_2	N_2	T_2	
3	W_3	N_3	T_3	
⋮				
n	W_n	N_n	T_n	
		ΣN	ΣT	$\Sigma c \cdot \Delta L$

The F.S. is then calculated from equation

$$\text{F.S.} = \frac{\Sigma c \cdot \Delta L + (\Sigma N - \Sigma U) \tan\phi}{\Sigma T} \qquad \ldots(20.25a)$$

20.13.1. Location of the Centre of the Critical Slip Circle. In order to find out the worst case, numerous slip circles should be assumed and factor of safety (F.S.)

calculated for each circle, as explained earlier. The minimum factor of safety will be obtained for the critical slip circle. In order to reduce the number of trials, Fellenius has suggested a method of drawing a line (PQ), representing the locus of the critical slip circle.

The determination of this line PQ for the d/s slope of an embankment is shown in Fig. 20.22(a) and similarly, the determination of this line PQ for u/s slope is shown in Fig. 20.22(b). The point Q is determined in such a way that its coordinates are ($4.5H$, H) from the toe, as shown. The point P is obtained with the help of directional angles α_1 and α_2 as shown. The value of α_1 and α_2 for various slopes, are tabulated in Table 20.7.

Table 20.7

Slope	*Directional angles*	
	α_1 *in degrees*	α_2 *in degrees*
1 : 1	27.5	37
2 : 1	25	35
3 : 1	25	35
4 : 1	25	35
5 : 1	25	35

After determining the locus of the critical slip circle, the critical slip circle can be drawn, keeping in view the following few points :

(*i*) Except for very small values of ϕ, the critical arc passes through the toe of the slope.

(*ii*) If a hard stratum exists at a shallow depth under the dam, the critical arc cannot cross this stratum, but can only be tangential to it.

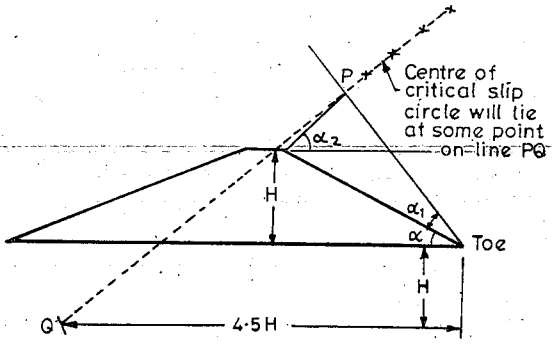


Fig. 20.22 (a). Locus of critical circle for d/s slope.

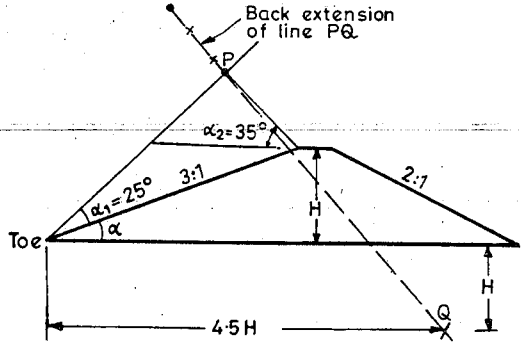


Fig. 20.22 (b) Locus of critical circle for u/s slope.

(*iii*) For very small values of ϕ (0 to 15°), the critical arc passes below the toe of the slope if the inclination of the slope is less than 53° (which is generally the case). The centre of the critical arc in such a case is likely to fall on a vertical line drawn through the centre of the slope, as shown in Fig. 20.23.

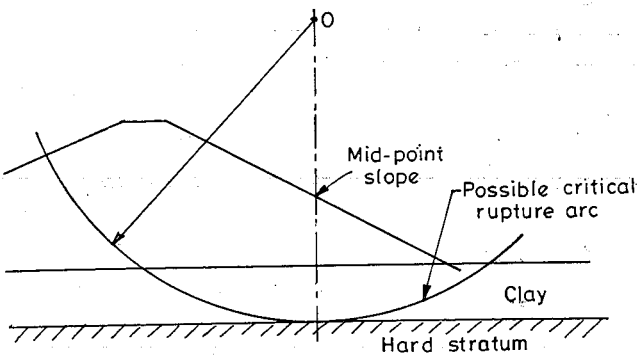


Fig. 20.23.

20.13.2. Determination of Pore Pressures from Flow Net. It was stated earlier that the soil is composed of voids which are filled with water and air. Whenever, any load is applied in the form of consolidation, it is taken up by water and gradually transferred to soil grains as the excess water drains out. The shear strength of the soil thus goes on increasing.

Immediately after construction, sizable pore pressures may be present, which may gradually dissipate. But as soon as the reservoir is filled, water enters the pores of the dams and a new pattern of pore pressure gets developed.

Under steady state seepage, the pore pressure at any point in a dam is equal to the hydrostatic head due to water in the reservoir minus the head loss in seepage through the dam up to that point. The pore pressure at any point within a dam section can easily be found from the flow net.

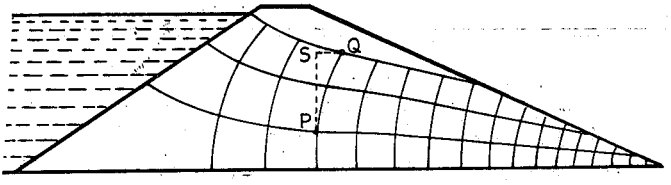


Fig. 20.24

For example, in Fig. 20.24, the pore pressure at any point say P will be equal to the difference in elevation between P and Q, where Q is the point of intersection of the phreatic line with the equipotential line through the point P.

Hence, if a piezometer is installed at F, the water shall rise up to an elevation of Q. If QS is a horizontal line through Q, PS will be the pore pressure at P.

20.13.3. Stability of Downstream Slope during Steady Seepage. The most critical condition for which the stability of the d/s slope must be examined, occurs, when the reservoir is full and the seepage is taking place at full rate.

The seeping water below the phreatic line, exerts a pore pressure on the soil mass which lies below the phreatic line. Hence, if the slices of the critical arc, happen to include this submerged soil, [Fig. 20.25 (*a*)], the shear strength developed on those slices shall be correspondingly reduced. The net shear strength on such a slice shall be $= c\,\Delta L + (N - U)\tan\phi$, where U is the pore pressure.

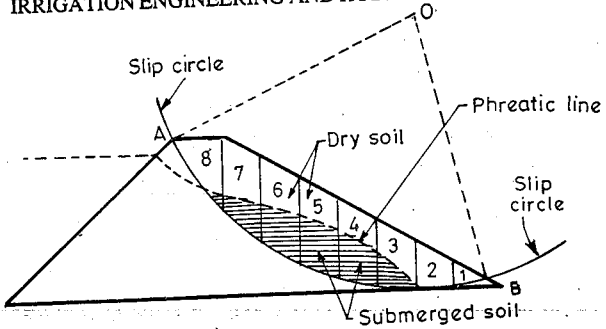


Fig. 20.25 (*a*)

The factor of safety (S.F.) for the entire slip circle is then given by the equation.

$$\text{F.S.} = \frac{c \cdot \widehat{AB}. + \tan\phi\,(\Sigma N - \Sigma U)}{\Sigma T} \qquad ...(20.25)$$

where ΣU is the total pore pressure on the slip circle.

The pore pressure at a point is represented by the piezometric head at that point as explained earlier. The variation of the pore pressure along a failure arc is, therefore, obtained as explained below :

First of all, draw a flow net and thus determine the points of intersections of equipotential lines with the failure arc. At each point of intersection, measure the vertical ordinate from that intersection to the level at which that particular equipotential line cuts the phreatic line. The pore pressures represented by the vertical heights so obtained, are then plotted to a scale in a direction perpendicular to the sliding surface at the respective points of intersection.

The pore pressure distribution is thus shown in Fig. 20.25(*b*) (shaded area). The

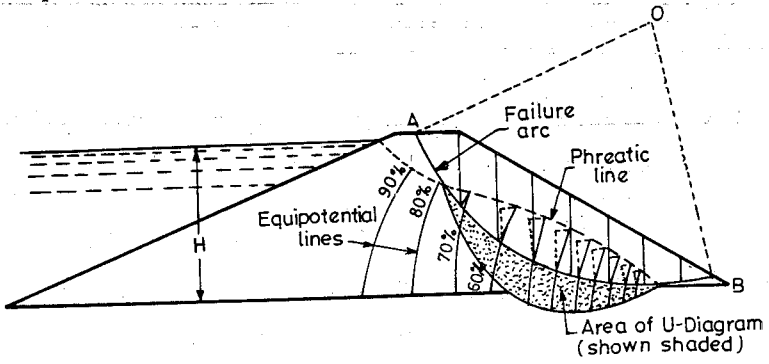


Fig. 20.25 (*b*)

area of this diagram can be measured by a planimeter. The area of this diagram can also be calculated by ordinate method as was done for N and T cases taking the unit weight of water as 9.81 kN/m^3 ($\approx$ 10 kN/m^3). Knowing ΣN, ΣU and ΣT, F.S. can be calculated easily by using equation 20.25.

Approximate method. In the absence of a flow net, the normal components, which are responsible for the shear strength of the soil, can be calculated on the basis of submerged unit weight of soil (*i.e.* $\gamma_{sub} = \gamma_{sat} - \gamma_w$). On the other hand, the values of

tangential components *i.e.* T's which are responsible for creating disturbing moments, should be calculated on the basis of saturated unit weight of soil.

In such a case, the width of N rectangle which was taken as $b \cdot \gamma \cdot \cos \alpha$ will become $b \cdot \gamma_{sub} \cos \alpha$ (Fig. 20.21) and the width of T rectangle will remain $b \cdot \gamma_{sat} \cdot \sin \alpha$. The new N diagram is a $(N - U)$ diagram assuming the entire soil to be submerged and can be called N' diagram. Equation (20.25) can be written as

$$\text{F.S.} = \frac{c \cdot \widehat{AB} + \tan \phi \cdot (\Sigma N')}{\Sigma T} \qquad ...(20.26)$$

20.13.4. Stability of Upstream Slope During Sudden Drawdown. When the reservoir is full, the critical region is near the downstream face. If no drainage arrangement is made and the d/s slope is also steep, the phreatic line may intersect the d/s slope creating serious conditions there. This can be avoided by providing drainage filter or drainage toe, etc., or by broadening the base of the dam so that the head loss is great enough to bring the line of saturation beneath the d/s toe of the dam.

For the u/s slope, the critical condition can occur, when the reservoir is suddenly emptied. In such a case, the water level within the soil will remain as it was when the soil pores were full of water. The weight of this water within the soil, now tends to slide the u/s slope along a circular arc.

The tangential component of the saturated soil lying over the arc, will create a disturbing force ; while the normal component minus the pore pressure shall supply the shear strength of the soil. High pore pressures shall be developed in this case and although a true solution can be obtained from the flow net and pressure net, an approximate solution can be easily obtained, by considering the soil resting over the failure arc as saturated, for calculating T's ; and as submerged for calculating N's.

The factor of safety (F.S.) is finally obtained from the equation

$$\text{F.S.} = \frac{c \cdot \widehat{AB} + \tan \phi \, \Sigma N'}{\Sigma T}$$

N's represent normal components on submerged density and T's represent tangential components on saturated unit weight of soil. The maximum factor of safety obtained for the critical slip circle should be 1.5, for safe designs.

20.13.5. Stability of the u/s slope portion of the dam, during sudden drawdown, from the consideration of horizontal shear developed at base under the u/s slope of the dam. It is an approximate method for checking the stability of the u/s slope against sudden drawdown. It is based on the simple principle that a horizontal shear force (say P_u) is exerted by the saturated soil (*i.e.* by the soil as well as by water contained within the soil). The resistance to this force (say R_u) is provided by the shear resistance

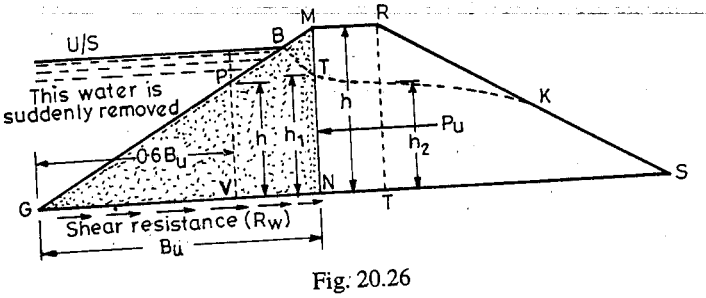


Fig. 20.26

developed at the base of the soil mass, contained within the u/s triangular shoulder *GMN* (Fig. 20.26).

Considering a unit length of the dam, the horizontal force P_u is given by the equation

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^2}{2}\right] \text{(say in kN)} \quad ...(20.27)$$

where γ_1 is the weighted density at the centre of triangular shoulder upstream and is given by

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry} \cdot (h - h_1)}{h} \quad ...(20.28)$$

Shear resistance (R_u) of u/s slope portion of the dam developed at base GN is given by

$$R_u = C + W \tan \phi \quad ...(20.29a)$$

where W = the weight of the u/s triangular shoulder of dam

C = The total cohesive force developed at base *GN*.

If c is the unit cohesion of the dam soil, then $C = c \times (B_u \times 1)$ where B_u = length *GN*.

The triangular profile of the u/s slope portion of dam has an area *GBTN* as the submerged soil (*i.e.* the soil below the seepage line) and an area equal to *BMT* as a dry area. The correct weight W will be equal to ($\gamma_{sub} \times$ Area *GBTN* + $\gamma_{dry} \times$ Area *BMT*). These areas can be measured by a planimeter. If the measuring of the areas is to be avoided, the entire area may be taken as submerged. By so doing, the weight W will be slightly reduced, and thus $W \tan \phi$ or R_u or F.S. will be slightly reduced. Hence, the results obtained will be on a safer side.

In such a case,

$$W = [\text{Area of } \Delta GMN] \cdot \gamma_{sub} = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot Bu \cdot h)$$

or

$$R_u = C + W \tan \phi$$
$$= c \cdot (B_u \times 1) + (\gamma_{sub} \cdot \tfrac{1}{2} \cdot B_u \cdot h) \tan \phi \quad ...(20.29)$$

Now P_u and R_u are known, the factor of safety against sliding can be easily calculated, using

$$\text{F.S.} = \frac{R_u}{P_u} \quad ...(20.30)$$

It should be more than 1.5.

The factor of safety calculated above is w.r. to average shear (τ_{av}), which will be equal to

$$\tau_{av} = \left(\frac{P_u}{B_u \times 1}\right) \quad ...(20.31)$$

It has been found by photoelastic studies, that the maximum intensity of shear stress occurs at a distance 0.6 B_u from the heel (*i.e.* 0.4 B_u from the top shoulder) and is equal to 1.4 times the average shear intensity.

$\therefore$ Maximum shear stress induced

$$= \tau_{max} = 1.4 \left(\frac{P_u}{B_u} \right) \qquad ...(20.32)$$

which is developed at the point V (Fig. 20.26) such that $GV = 0.6\, B_u$.

The unit shearing resistance developed at this point V is give by

$$\tau_f = c + h' \gamma_{sub} \cdot \tan \phi$$
$$= c + 0.6\, h \cdot \gamma_{sub} \cdot \tan \phi \qquad ...(20.33)$$

F.S. at the point of the maximum shear

$$= \frac{\tau_f}{\tau_{max}} \qquad ...(20.34)$$

It should be more than 1.

20.13.6. Stability of d/s slope under steady seepage from the considerations of horizontal shear at base under the d/s slope of the dam. The stability of the d/s slope under steady seepage is generally tested with Swedish slip circle method. However, the F.S. against the horizontal shear forces can be evaluated on the same principles as was done for the d/s slope in the previous article.

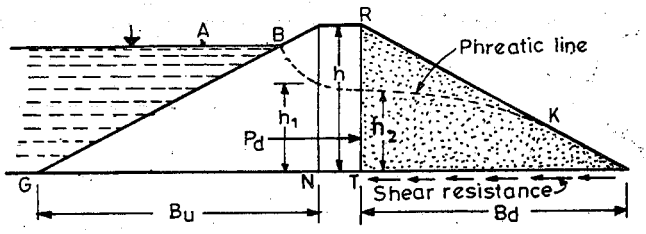


Fig. 20.27

With reference to Fig. 20.27.

The horizontal shear force P_d, in this case, is given by

$$P_d = \left[\frac{\gamma_2 \cdot h^2}{2} \tan^2 \left(45 - \frac{\phi}{2} \right) + \frac{\gamma_w \cdot h_2^2}{\cdot 2} \right] \qquad ...(20.35)$$

where γ_2 is the weighted density at the centre of the triangular shoulder downstream, given by

$$\gamma_2 = \frac{\gamma_{sub} \cdot h_2 + \gamma_{dry} (h - h_2)}{h} \qquad ...(20.36)$$

Shear resistance R_d of d/s slope portion of dam, developed at base is given by

$$R_d = C + W \tan \phi,$$

where W = The weight of the d/s slope portion of dam. (*i.e.* $\Delta\, RTS$)

$C = c \times (B_d \times 1)$, where c is the unit cohesion.

The triangular profile RTS of the d/s slope portion of dam has an area say A_1 of dry soil above the seepage line and the area of submerged soil say A_2 below the seepage lines. These areas can be measured by a planimeter and then

$$W = [\gamma_{dry} A_1 + \gamma_{sub} \cdot A_2] \times 1$$

or

$$R_d = c \cdot B_d + \left[\gamma_{dry} A_1 + \gamma_{sub} \cdot A_2 \right] \tan \phi \qquad ...(20.37)$$

If the measuring of the areas is to be avoided, the entire weight W may be calculated on the basis of submerged soil, as it will be on a still safer side. In that case,

$$W = \gamma_{sub} \cdot (\tfrac{1}{2} \cdot B_d \cdot h)$$

Knowing P_d and R_d, the factor of safety against shear can be easily determined as

$$\text{F.S.} = \frac{R_d}{P_d} \qquad ...(20.38)$$

The factor of safety at the point of maximum shear can also be determined in the same manner as was explained for the u/s slope portion.

20.13.7. Stability of the foundation against Shear. The generally available silt and clay foundations below the base of an earth dam, are sufficiently impervious, and there is generally no necessity of providing any treatment for under-seepage and piping for such foundations. But these foundations are weak in shear and must be investigated. In order to keep the shear stress developed at the foundations, within limits, the embankment-slopes may have to be flattened or berms on either side may be provided. If the available foundations are of plastic or unconsolidated clays, their shear strength will be very less and the matter should be seriously and thoroughly investigated.

The method given below, for determining the factor of safety against the foundation shear, is an approximate method and is based on the assumption that a soil has an equivalent liquid unit weight which would produce the same shear stress as the soil itself.

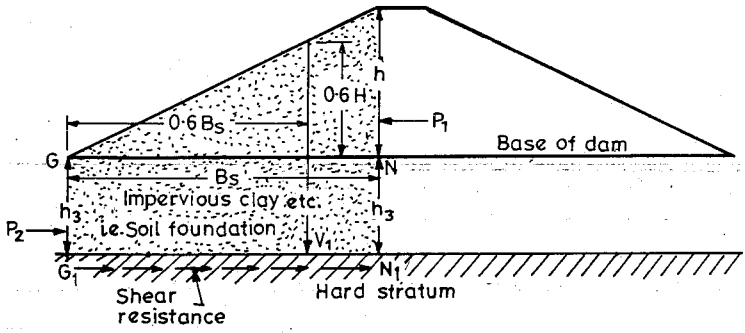


Fig. 20.28

The total horizontal shear force (P), under a slope of the dam is equal to the difference between the lateral thrust on a vertical through the top shoulder of the slope and a vertical through the toe of the slope.

$$\text{or} \qquad P = (P_1 - P_2) = \frac{\gamma_{eq}\,(h + h_3)^2}{2} \tan^2\left(45 - \frac{\phi_1}{2}\right) - \frac{\gamma_{eq} \cdot h_3^{\,2}}{2} \times \tan^2\left(45 - \frac{\phi_1}{2}\right)$$

$$\text{or} \qquad P = \gamma_{eq}\left[\frac{(h + h_3)^2 - h_3^{\,2}}{2}\right] \times \left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right] \quad \text{(kN)} \qquad ...(20.39)$$

where h_3 = the depth of foundation soil below the dam base, overlying the hard stratum below it.

γ_{eq} = the equivalent unit weight of dry soil in foundation and dam. It is given by

$$\gamma_{eq} = \frac{\gamma_{dry} \text{ for dam material} \times h + \gamma_{dry} \text{ for foundation material} \times h_3}{h + h_3} \quad ...(20.40)$$

where ϕ_1 = the equivalent angle of internal friction and is given by

$$\phi_1 = \tan^{-1}\left[\frac{c_f + \gamma_{eq}(h + h_3)\tan\phi_f}{\gamma_{eq} \cdot (h + h_3)}\right] \quad ...(20.41)$$

where c_f and ϕ_f are the values of unit cohesion and angle of internal friction for the soil in the foundation.

The term, $\left[\gamma_{eq} \cdot \tan^2\left(45 - \frac{\phi_1}{2}\right)\right]$ is known as **equivalent liquid unit weight.**

Now the average shear stress at the base of the slope

$$(\tau_{av}) = \frac{P}{B_s} \quad ...(20.42)$$

where B_s is the base width below the slope.

The value of B_s will be equal to B_u for u/s slope and B_d for d/s slope. The minimum value will generate maximum stresses and hence, that particular slope should be considered which gives the minimum value of B_s, *i.e.* the slope which is less flat and is, therefore, the worst slope.

Maximum stress has been found by photoelastic studies to be 1.4 times the average stress and it occurs at a distance of 0.6 B_s from the toe of the slope.

or τ_{max} = maximum shear stress = 1.4 τ_{av}.

The unit shear resistance of the foundation soil below the toe at point G_1

$$= \tau_{f_1} = \left[c_f + \gamma_f \cdot h_3 \tan\phi_f\right] \quad ...(20.43)$$

γ_f is the unit weight of foundation soil and if the average value is given for impervious soils, that value may be used in the equation (20.43). But if there is a possibility of foundation soil getting submerged due to large scale seepage that may take place through the foundation soil, then the submerged density may be used in equation (20.43).

Similarly, the unit shear resistance of the soil vertically below the upper point of the considered slope (say at point N_1) is given by τ_{f_2}.

$$\tau_{f_2} = c_f + \gamma_3 (h + h_3) \tan\phi_f \quad ...(20.44)$$

where γ_3 = the equivalent unit weight of soil in the dam and foundation at the point N_1 and is given by

$$\gamma_3 = \frac{\gamma_f \times h_3 + \gamma_{dam} \times h}{h + h_3} \quad ...(20.45)$$

The values of γ_f and γ_{dam} may be taken as their dry densities or submerged densities depending upon the possibilities.

The average shear resistance

$$= \tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} \quad ...(20.46)$$

Hence, overall factor of safety $= \dfrac{\tau_f}{\tau_{(av)}}$ ...(20.47)

This should be greater than 1.5.

The factor of safety at the point of maximum shear (*i.e.* the point V_1) must also be calculated as below :

The unit shear resistance at this point V_1

$$= \tau_{f(max)} = c_f + \gamma_4 (h_3 + 0.6h) \tan \phi_f$$

where γ_4 is the equivalent weight of soil in dam and foundation and is given by

$$\gamma_4 = \frac{\gamma_f \times h_3 + \gamma_{dam} \times (0.6\, h)}{(h_3 + 0.6h)} \quad ...(20.48)$$

The dry or submerged unit weights may be used as explained earlier, in the above equation.

F.S. = Factor of safety at the point of maximum shear

$$= \frac{\tau_{f(max)}}{\tau_{max}} \quad ...(20.49)$$

This should be greater than unity.

Example 20.4. *An earthen dam made of homogeneous material has the following data :*

Level of the top of the dam	$= 200.00$ *m*
Level of deepest river bed	$= 178.0$
H.F.L. of reservoir	$= 197.5$ *m*
Width of top of dam	$= 4.5$ *m*
Upstream slope	$= 3 : 1$
Downstream slope	$= 2 : 1$
Length of the horizontal filter from d/s toe, inwards	$= 25$ *m*
Cohesion of soil of dam	$= 24$ *kN/m*^{2}
Cohesion of soil of foundation	$= 54$ *kN/m*^{2}
Angle of internal friction of soil in the dam	$= 25°$
Angle of internal friction of soil in the foundation	$= 12°$
Dry weight of the soil in the dam	$= 18$ *kN/m*^{3}
Submerged weight of the soil in the dam	$= 12$ *kN/m*^{3}
Dry unit weight of the foundation soil	$= 18.3$ *kN/m*^{3}
Coefficient of permeability of soil in the dam	$= 5 \times 10^{-6}$ *m/sec.*

The foundation soil consists of 8 m thick layer of clay, having negligible coefficient of permeability. Check the stability of the dam and its foundations.

Solution.

(1) Overall stability of the dam section as a whole

We will consider 1 m length of the dam. The section of the dam and the phreatic line is first of all drawn, as given in example 20.3 and shown in Fig. 20.29 (*a*). The dam section, etc. is generally drawn on a graph sheet so as to facilitate in measuring the areas above and below the seepage line, if planimeter is not available.

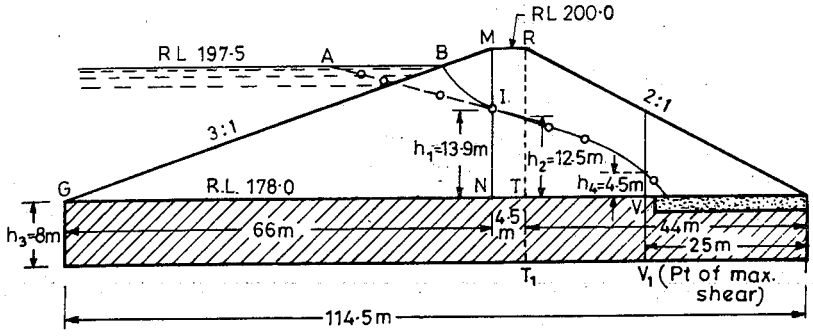


Fig. 20.29 (*a*)

The total area of dam section $= (114.5 + 4.5)\dfrac{22}{2} = 1{,}409$ sq. m

The area above the seepage line is measured and is approximately found to be 380 m^2. (In the absence of a planimeter, graph can be used).

$\therefore$ Area below the seepage line $= 1{,}409 - 380 = 1{,}029$ sq. m

Now

Weight of the dry portion of the dam section

$$= (380\ m^2 \times 1\ m \times 18\ kN/m^3 = 6830\ kN.$$

Weight of the submerged portion of the dam section

$$= 1029\ m^2 \times 1\ m \times 12\ kN/m^3 = 12{,}350\ kN$$

Total weight of dam (called average weight)

$$= 6{,}830 + 12{,}350 = 19{,}180\ kN$$

Shear resistance of the dam at the base

$$= C + W \tan \phi$$

where C = Total cohesive strength of the soil at the base

$= c \times B \times 1 = (24 \times 114.5 \times 1)$ kN

B = Total base width = 114.5 m

$W \tan \phi = 19{,}180 \tan 25°$

$\therefore$ Shear resistance at base,

$$R = 24 \times 114.5 \times 1 + 19180 \tan 25° = 11690\ kN$$

Horizontal force = Horizontal pressure of water.

$$= P = \tfrac{1}{2}\gamma_w h^2 = \tfrac{1}{2} \cdot 9.81\ (19.5)^2 = 1865\ kN$$

Factor of safety against failure due to horizontal shear at base

$$= \frac{11690}{1865} = 6.27 > 1.3\ (\therefore \text{Safe})$$

(2) **Stability of the u/s slope portion of dam (Under sudden drawdown)** *horizontal shear along the base under the u/s slope of dam*

Draw a vertical through the u/s extremity of the top width of dam [*i.e.* point *M*, Fig. 20.29 (*a*)] so as to cut the base of the dam at point *N*. This vertical *MN* cuts the seepage

line at a point, the height of which is measured as $h_1 = 13.6$ m above the base of the dam.

Horizontal force (P_u) acting on the ΔGMN is given by equation (20.27) as :

$$P_u = \left[\frac{\gamma_1 h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \gamma_w \cdot \frac{h_1^2}{2}\right]$$

where γ_1 = the weighted density at the centre of triangular shoulder upstream (ΔGMN) and is given by equation (20.28) as :

$$\gamma_1 = \frac{\gamma_{sub} \cdot h_1 + \gamma_{dry}(h - h_1)}{h}$$

$$= \frac{12 \times 13.9 + 18\,(22.0 - 13.9)}{22.0}$$

$$= 14.7 \text{ kN/m}^3$$

$$\therefore \quad P_u = \frac{14.7 \times (22.0)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \times \frac{(13.9)^2}{2} = 2391 \text{ kN}$$

Shear resistance R_u of the u/s slope portion of dam developed at the base GN is given by equation (20.29) as :

$$R_u = C + W \tan \phi$$

$$= c\,(B_u \times 1) + (\gamma_{sub} \tfrac{1}{2} B_u h) \tan \phi$$; neglecting the small dry soil area BMI, as it is very small and this neglection is on a safer side.

$$B_u = 66 \text{ m}$$

$$\therefore \quad R_u = 24 \times 66 + (12 \cdot \tfrac{1}{2} \cdot 66 \cdot 22.0) \tan 25°$$

$$= 1584 + 4062 = 5646 \text{ kN}$$

Factor of safety against horizontal shear along base under u/s slope

$$= \frac{R_u}{P_u} = \frac{5646}{2391} = 2.36 > 2.0 \qquad (\therefore \text{ safe})$$

Horizontal shear stress induced in the u/s slope portion of dam at base.

$$\tau_{av} = \frac{P_u}{B_u \times 1} = \frac{2391}{66} \text{ kN/m}^2 = 36.23 \text{ kN/m}^2$$

τ_{max} = Maximum shear

$$= 1.4\, \tau_{av} = 1.4 \times 36.23 = 50.72 \text{ kN/m}^2$$

The maximum shear is developed at a point $0.6\, B_u$

$$= 0.6 \times 66 = 39.6 \text{ m away from point } G$$

The unit shear resistance developed at this point

$$\tau_f = c + 0.6\, \gamma_{sub} \tan \phi$$

$$= 24 + 0.6 \times 22.0 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} = \frac{97.9}{50.72} = 1.93 > 1 \qquad (\therefore \text{ safe}).$$

(3) **Stability of d/s portion of dam.** *Horizontal shear along base under the d/s slope of dam.*

Draw a vertical through the d/s extremity of the top width of dam (*i.e.* point *R*) to cut the base at point *T* [Fig. 20.29 (*a*)]. Let this vertical cut the seepage line in a point, the height of which from the base is measured as $h_2 = 12.5\ m$.

Horizontal force P_d acting on the portion of downstream dam (*RTS*) during steady seepage is given by equation (20.35) as :

$$P_d = \left[\frac{\gamma_2 h^2}{2} \tan^2\left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h_2^2}{2}\right]$$

where γ_2 is the weighted density at the centre of the triangular shoulder *RTS* and given by equation (20.36) as :

$$\gamma_2 = \frac{\gamma_{sub}\, h_2 + \gamma_{dry}\,(h - h_2)}{h}$$

$$= \frac{12 \times 12.5 + 18 \times (22.0 - 12.5)}{22.0}$$

$$= 14.6\ \text{kN/m}^3$$

$$P_d = \frac{14.6\,(22)^2}{2} \tan^2\left(45° - \frac{25°}{2}\right) + 9.81 \cdot \frac{(12.5)^2}{2} = 2200\ \text{kN}$$

Shear resistance R_d of the d/s slope portion of dam developed at base *TS* is given as :

$$R_d = C + W \tan \phi$$

The area A_1 of the dry soil within the $\Delta\, RTS$ above the seepage line ≈ 300 sq. m (from graph or planimeter).

The total area of the $\Delta\, RTS = \frac{1}{2} \times 44 \times 22 = 484\ \text{m}^2$

$\therefore$ Area of submerged soil

$$A_2 = 484 - 300 = 184\ \text{sq. m}$$

$$R_d = cB_d + \left[\gamma_{dry} A_1 + \gamma_{sub} A_2\right] \tan \phi$$

$$= 24 \times 44.0 + [18 \times 300 + 12 \times 184] \tan 25° = 4604\ \text{kN}.$$

F.S. against horizontal shear along base under d/s slope

$$= \frac{R_d}{P_d} = \frac{4604}{2200} = 2.09 > 2\ (\therefore\ \text{Safe})$$

Average shear induced at base

$$= \frac{P_d}{B_d} = \frac{2200}{44} = 50\ \text{kN/m}^2$$

Maximum shear stress induced

$$\tau_{max} = 1.4 \times 50 = 70\ \text{kN/m}^2$$

The maximum shear stress is developed at a point $0.6\ B_d$

$$= 0.6 \times 44 = 26.4\ \text{m away from toe}$$

This unit shear resistance developed at this point

$$\tau_f = c + 0.6h\,\gamma_{sub}\tan\phi$$

(assuming the entire height as submerged as it will give safer results)

$$= 24 + 0.6 \times 22 \times 12 \tan 25° = 97.9 \text{ kN/m}^2$$

$$\therefore \quad \text{F.S.} = \frac{\tau_f}{\tau_{max}} \frac{97.9}{70} = 1.40 > 1 \qquad (\therefore \text{ Safe})$$

(4) **Stability of the foundation soil**

Average compressive stress on foundation soil

$$= \frac{\text{Weight of dam}}{\text{Base area on which it acts}}$$

Since the compressive stress is maximum when the entire dam soil is dry, therefore, we will first calculate the dry weight of the dam.

Area of section of dam

= 1,409 sq. m (calculated earlier)

Dry weight of dam section

$$= 18 \times 1{,}409 = 25{,}362 \text{ kN}$$

Average compressive stress at base

$$= \frac{25362}{114.5} = 221.5 \text{ kN/m}^2$$

Shear stress induced at base

The total horizontal shear force (P) under the d/s slope of the dam (which is the worst case, *i.e.* the steepest slope) is given by equation (20.39) as :

$$P = \gamma_{eq}\left[\frac{(h+h_3)^2 - h_3^2}{2}\right]\left[\tan^2\left(45° - \frac{\phi_1}{2}\right)\right]$$

where γ_{eq} is the equivalent weight of dry soil in foundation and dam

$$\gamma_{eq} = \frac{18h + 18.3\,h_3}{h + h_3}$$

[$\because$ Unit wt. of foundation soil of thickness $h_3 = 18.3 \text{ kN/m}^3$]

where $h = 22$ m

$h_3 = 8$ m.

$$\therefore \quad \gamma_{eq} = \frac{18 \times 22 + 18.3 \times 8}{22 + 8} = 18.1 \text{ kN/m}^3$$

ϕ_1 is given by equation (20.41) as :

$$\gamma_{eq}(h + h_3)\tan\phi_1 = c_f + \gamma_{eq}(h + h_3)\tan\phi_f$$

or $$18.1\,(22 + 8)\tan\phi_1 = 54 + 18.1\,(22 + 8)\tan 12°$$

or $$\tan\phi_1 = 0.312$$

or $$\phi_1 = 17.3°$$

$$\therefore \quad P = 18.1\left[\frac{(22+8)^2 - (8)^2}{2}\right][\tan^2(45° - 8.65°)]$$

$$= \frac{18.1}{2} [900 - 64] [(0.737)^2] = 4100 \text{ kN}$$

Average shear stress induced at base of d/s slope

$$\tau_{av} = \frac{4100}{44} = 93.2 \text{ kN/m}^2$$

Maximum shear stress induced at $0.6 \times 44 = 26.4$ m away from the d/s toe inwards at point V_1 is given by

$$= \tau_{max} = 1.4 \times 93.2 = 130.4 \text{ kN/m}^2$$

Shear resistance of the foundation soil below the d/s slope portion of dam

Unit shear resistance τ_{f_1} below the toe at point S_1

$$= [c_f + \gamma_f \times h_3 \tan \phi_f]$$

$$= 54 + 18.3 \times 8 \times \tan 12°$$

$$\therefore \quad = 85.1 \text{ kN/m}^2$$

Unit shear resistance τ_{f_2} at point T_1

$$= c_f + \gamma_3 (h + h_3) \tan \phi_f$$

where $$\gamma_3 = \frac{\gamma_{sub\,for\,dam} \times h_2 + \gamma_{dry\,for\,dam} \times (h - h_2) + \gamma_f h_3}{h + h_3}$$

$$= \frac{12 \times 12.5 + 18 \times 9.5 + 18.3 \times 8}{30} = 15.6 \text{ kN/m}^3$$

$$\therefore \quad \tau_{f_2} = 54 + 15.6 (22 + 8) \times \tan 12° = 153.5 \text{ kN/m}^2$$

The average unit shear resistance developed at foundation level in a length equal to $T_1S_1 = 44$ m, is given by

$$\tau_f = \frac{\tau_{f_1} + \tau_{f_2}}{2} = \frac{85.1 + 153.5}{2} = 119.3 \text{ kN/m}^2$$

Over all F.S. against shear

$$= \frac{\tau_f}{\tau_{av}} = \frac{119.3}{93.2} = 1.28 < 1.5 \qquad \textbf{(Hence, unsafe)}$$

The foundation soil is thus weaker to carry the load and hence the d/s slope will have to be flattened.

Shear resistance at the point of maximum shear, *i.e.* at point V_1 is given as :

$$(\tau_f)_{max} = c_f + (0.6h + h_3) \gamma_4 \tan \phi_f$$

$$\gamma_4 = \frac{12 \times 4.5 + 18 (0.6 \times 22 - 4.5) + 18.3 \times 8}{0.6 \times 22 + 8} = 16.8 \text{ kN/m}^3$$

$$(\tau_f)_{max} = 54 + [0.6 \times 22 + 8]\, 16.8 \tan 12° = 129.8 \text{ kN/m}^2$$

$$\text{F.S.} = \frac{[\tau_f]_{max}}{\tau_{max}} = \frac{129.8}{130.4} = 0.995 < 1.0 \qquad \text{(Hence, Unsafe)}$$

The foundation shear and F.S. can also be calculated below the u/s portion of dam soil, in the same manner as has been done for d/s slope portion, if required.

Example 20.5. *Check the stability of the d/s slope of the earthen dam section, given in example 20.4, on a possible slip circle. Determine the factor of safety available*

against such a slip. Also determine the net factor of safety that will be available, if by some how, the soil in the d/s triangular shoulder gets fully submerged. Also compare this net factor of safety with the factor of safety that can be obtained for similar conditions if the analysis was done by shear force determination at the base of the d/s slope.

Solution. The dam section and phreatic line is first of all drawn as shown in Fig. 20.29 (*b*) and as given in previous example. The points *P* and *Q* are located for Fellenius construction such as explained earlier ; the line *PQ* represents the locus of the critical slip circle. A possible slip circular arc passing through the toe of the dam is then drawn with centre at O_1. The soil mass is then divided into slices of 5 metre width. It is found

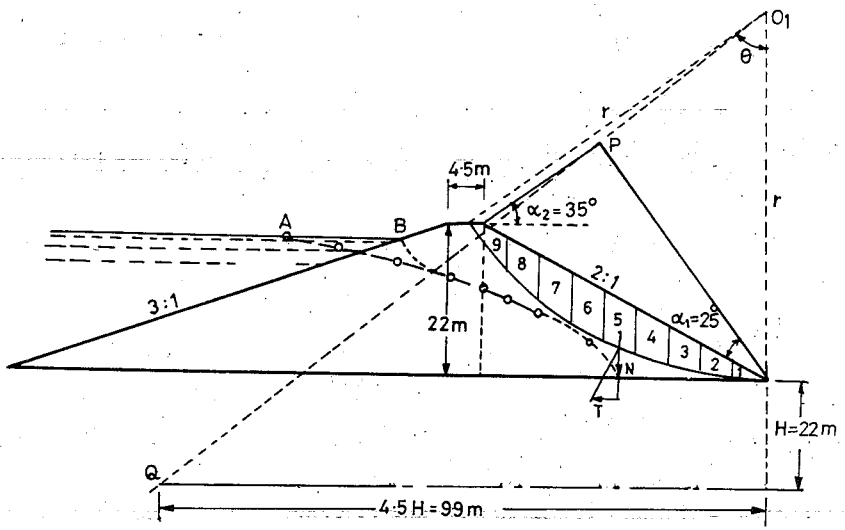


Fig. 20.29 (*b*)

by chance that full 9 slices of 5 m width are accommodated, making thereby the width of the last slice (which was *m.b*) equal to *b* or $m = 1$. By visual observations ; it is also found that the tangenital components (*T*'s) of the weights of all the slices (acting through their centres) happen to be +ve. This is because, the lines of action of all the weights are on one side (*i.e.* left side) of the centre O_1.

The end ordinates of all the slices say $y_1, y_2, y_3, \ldots, y_{n-1}$ or y_8 are measured and found to be 2.5 m, 4.5 m, 6 m, 7 m, 7.5 m, 6.0 m, and 4.5 m respectively.

The *N*-diagram is then drawn as shown in Fig. 20.30 (*a*). The dry density is used because the entire soil in the slices is dry.

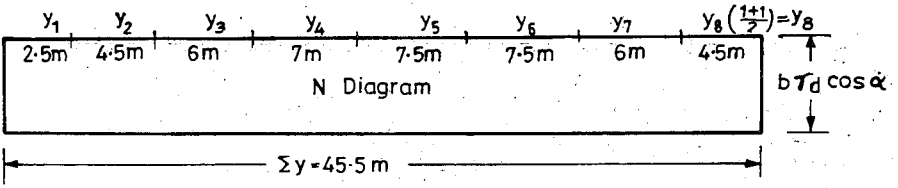


Fig. 20.30 (*a*). *N*-diagram.

Area of N-Diagram $= 45.5 \times 5 \times 18 \times \frac{2}{\sqrt{5}} = 3663$ kN

Similarly, T-Diagram is drawn as shown in Fig. 20.30 (b).

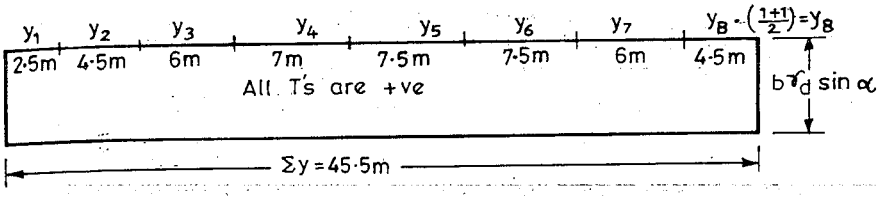


Fig. 20.30 (b). T-diagram.

Area of T-diagram $= 45.5 \times 5 \times 18 \times \frac{1}{\sqrt{5}} = 1831$ kN

Since the slip-circle-arc happens to fall completely above the phreatic line, the pore pressure area is zero ; or area of U- diagram is zero.

$\therefore \quad \Sigma N - \Sigma U = \Sigma N = 3663$ kN.

Now $$\text{F.S.} = \frac{c \cdot \widehat{AB} + (\Sigma N - \Sigma U) \tan \phi}{\Sigma T}$$

where $$\widehat{AB} = \frac{2\pi \cdot r}{360^\circ} \times \theta^\circ$$

The angle AO_1B (θ) is measured and found to be 58°. The radius r of the curve AB is also measured and found to be 53.5 m.

$\therefore \quad \widehat{AB} = \frac{2\pi \times 53.5}{360^\circ} \times 58^\circ = 54$ m

$c = 24$ kN/m^2 (given in previous example)

Hence $$\text{F.S.} = \frac{24 \times 54 + 3663 \times \tan 25^\circ}{1831} = 1.64 > 1.5$$ **(Hence, Safe)**

The corresponding F.S. obtained in example 20.4 from shear analysis was 2.08. Hence, F.S. should be more than 1.5 in slip circle analysis and more than 2 in shear analysis for safe designs.

Note. The F.S. obtained above by slip circle method cannot be called as the critical F.S. or minimum F.S.; because the analysis has been carried out for only one circle. In fact, various other circles should be drawn by taking the centres on line PQ, somewhere near P or away from it. If the circle happens to pass through the foundation soil, then the values of c_f and ϕ_f (*i.e.* c and ϕ for foundation soil) should be used in evaluating W and $c \Delta L$ for those particular slices. A tabular form solution, as explained earlier would then be better. When the slices are crossing the phreatic line, U-diagram has to be drawn and evaluated as explained earlier.

Case (b). When soil in the slices get fully submerged, then N's will be calculated on the basis of submerged weights and Ts will be calculated on the basis of saturated weights.

$\therefore \quad \Sigma N = 45.5 \times 5 \times \gamma_{sub} \times \cos \alpha$

$= 45.5 \times 5 \times 12 \times \frac{2}{\sqrt{5}} = 244$

$$\Sigma T = 45.5 \times 5 \times \gamma_{sat} \sin \alpha$$
$$= 45.5 \times 5 \times 22 \times \sin 25° = 2115 \text{ kN}$$

Net $$\text{F.S.} = \frac{c \cdot AB + \Sigma N \tan \phi}{\Sigma T}$$
$$= \frac{24 \times 54 + 2442 \tan 25°}{2115} = 1.15$$

Net F.S. = 1.15 Ans. < 1.5 (∴ Unsafe)

Hence, if such a possibility of submergence occurs, the dam becomes unsafe.

Horizontal shear at base of d/s slope under complete submergence condition.

$$P_d = \frac{\gamma_{sub} h^2}{2} \tan^2 \left(45° - \frac{\phi}{2}\right) + \gamma_w \frac{h^2}{2}$$
$$= \frac{12 \times (22)^2}{2} \tan^2 (45° - 12.5°) + \frac{9.81 \times (22)^2}{2} = 3553 \text{ kN}$$
$$R_d = c \cdot B_d + \gamma_{sub} (\tfrac{1}{2} \cdot B_d \cdot h) \tan \phi$$
$$= 24 \times 44.0 + 12 \times \tfrac{1}{2} \times 44 \times 22 \times \tan 25° = 3764 \text{ kN}$$

$$\text{F.S.} = \frac{3764}{3553} = 1.06 < 1.5$$ (Hence, Unsafe)

F.S. with slip circle analysis = 1.09
F.S. with horizontal shear analysis = 1.06 **Ans.**

Conclusion. The dam becomes unsafe as soon as the soil in the downstream shoulder gets submerged.

SEEPAGE CONTROL IN EARTH DAMS

The water seeping through the body of the earthen dam or through the foundation of the earthen dam, may prove harmful to the stability of the dam by causing softening and sloughing of the slopes due to development of pore pressures. It may also cause piping either through the body or through the foundation, and thus resulting in the failure of the dam.

20.14. Seepage Control Through Embankments

Drainage filters called 'Drains' are generally provided in the form of (*a*) *rock toe* (*b*) *horizontal blanket* (*c*) *chimney drain*, etc. in order to control the seepage water. The provision of such filters reduces the pore pressure in the downstream portion of the dam and thus increases the stability of the dam, permitting steep slopes and thus affecting economy in construction. It also checks piping by migration of particles. These drains, consist of graded coarse material in which the seepage is collected and moved to a point where it can be safely discharged. In order to prevent movement of the fine material from the dam into the drain, the drain or filter material is graded from relatively fine on the periphery of the drain to coarse near the centre. A multi-layered filter, generally called **inverted filter** or *reverse filter* is provided as per the criteria suggested by Terzaghi for the design of such filters.

The various kinds of drains, which are commonly used are shown and described below :

20.14.1. Rock Toe or Toe Filter [Fig. 20.31 (*a*)]. The 'rock toe' consists of stones of size usually varying from 15 to 20 cm. A toe filter (graded in layers) is provided as a transition zone, between the homogeneous embankment fill and rock toe. Toe filter

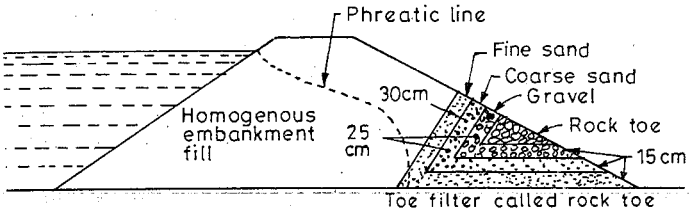


Fig. 20.31 (*a*). Rock Toe.

generally consists of three layers of fine sand, coarse sand, and gravel ; as per the filter criteria requirements. The height of the rock toe is generally kept between 25 to 35% of reservoir head. The top of the rock toe must be sufficiently higher than the tail water depth, so as to prevent the wave action of the tail water.

20.14.2. Horizontal Blanket or Horizontal Filter. [Fig. 20.31 (*b*) and (*c*)]. The horizontal filter extends from the toe (d/s end) of the dam, inwards, upto a distance varying from 25 to 100% of the distance of the toe from the centre line of the dam. Generally, a length equal to three times the height of the dam is sufficient. The blanket should be properly designed as per the filter criteria, and should be sufficiently pervious to drain off effectively.

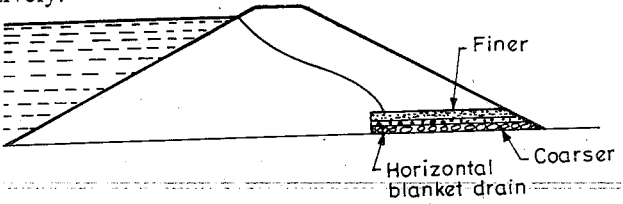


Fig. 20.31 (*b*). Horizontal Filter.

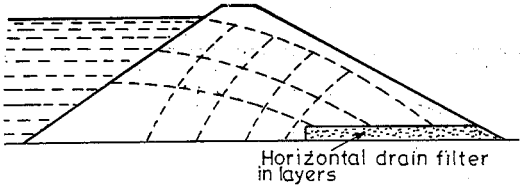


Fig. 20.31 (*c*). Inefficient 'Horizontal drain' in stratified embankments.

20.14.3. Chimney Drain. [Fig. 20.31 (*d*)]. The horizontal filter, not only helps in bringing the phreatic line down in the body of the dam but also provides drainage of the foundation and helps in rapid consolidation. But, the horizontal filter tries to make the soil more pervious in the horizontal direction and thus causes stratification. When large scale stratification occurs, such a filter becomes inefficient as shown in Fig. 20.30 (*c*). In such a possible case, a vertical filter (or inclined u/s or d/s) is placed along with the horizontal filter, so as to intercept the seep-

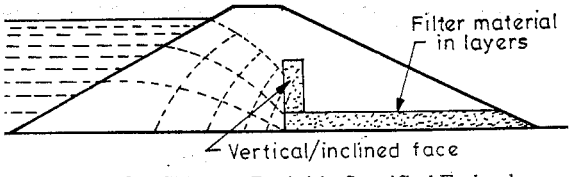


Fig. 20.31 (*d*). 'Chimney Drain' in Stratified Embankments.

ing water effectively, as shown in Fig. 20.31 (*d*). Such an arrangement is termed as *chimney drain.* Sometimes a horizontal filter is combined and placed along with a rock toe, as shown in Fig. 20.31 (*e*).

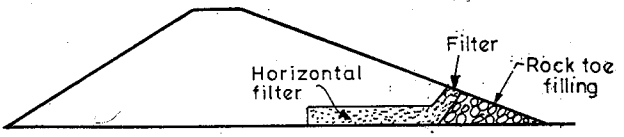


Fig. 20.31 (*e*). Horizontal filter combined with rock toe.

20.15. Seepage Control Through Foundations

The amount of water entering the pervious foundations, can be controlled by adopting the following measures :

20.15.1. Impervious Cutoffs. Vertical impervious cutoffs made of concrete or sheet piles may be provided at the upstream end (*i.e.* at heel) of the earthen dam (Fig 20.32). These cutoffs should, generally, extend through the entire depth of the pervious foundation, so as to achieve effective control on the seeping water. When the depth of the pervious foundation strata is very large, a cutoff, up to a lesser depth, called a *partial cutoff* may be provided. Such a cutoff reduces the seepage discharge by a smaller amount. So much so, that a 50% depth reduces the discharge by 25%, and 90% depth reduces the discharge by 65% or so.

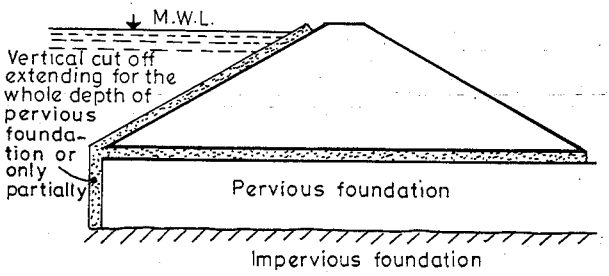


Fig. 20.32

20.15.2. Relief Wells and Drain Trenches. When large scale seepage takes place through the pervious foundation, overlain by a thin less pervious layer, there is a possibility that the water may boil up near the toe of the dam, as shown in Fig. 20.33 (*a*).

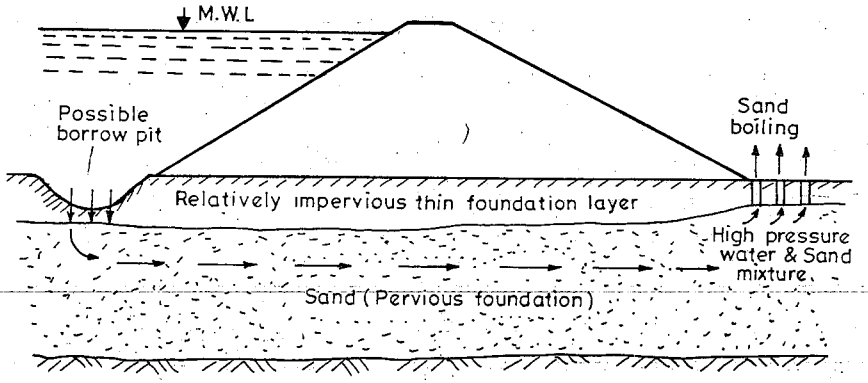


Fig. 20.33 (*a*). Sand Boiling Phenomenon.

Such a possibility, can be controlled by constructing relief wells or drain trenches through the upper impervious layer, as shown in Fig. 20.33 (*b*) and (*c*), so as to permit escape of seeping water. The possibility of sand boiling may also be controlled by

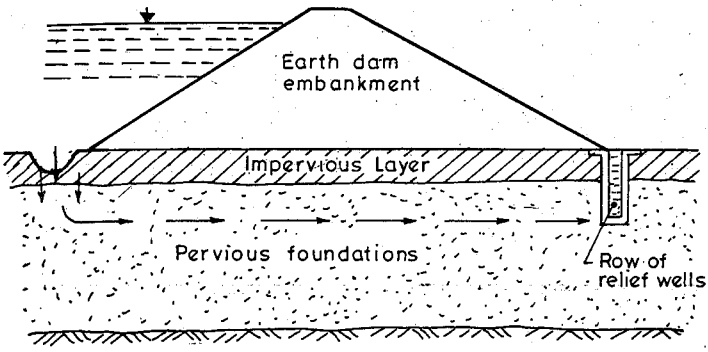


Fig. 20.33 (*b*). Provision of Relief Wells.

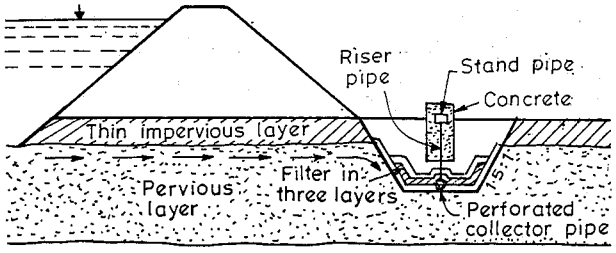


Fig. 20.33 (*c*). Enlarged View of Drain Trench.

providing d/s berms beyond the toe of the dam as shown in Fig. 20.33 (*d*). The weight of the overlying material, in such a case, is sufficient to resist the upward pressure and thus preventing the possibility of sand boiling. The provision of such berms, also protects the d/s toe from possible sloughing due to seepage.

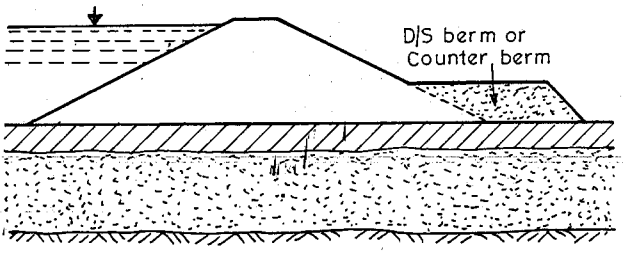


Fig. 20.33 (*d*). Provision of d/s Berms.

20.16. Design of Filters

The drainage filters must be designed in such a way that neither the embankment nor the foundation material can penetrate and clog the filters. The permeability or size of filter material should also be sufficient to carry the anticipated flow with an ample margin of safety. A rational approach to the design of filters has been provided by Terzaghi. According to him, the following filter criteria should be satisfied.

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of base materials}} < 4 \text{ to } 5 < \frac{D_{15} \text{ of filter}}{D_{15} \text{ of base material}} \qquad ...(20.50)$$

The embankment soil or the foundation soil surrounding the filter, is known as base material.

When the ratio of D_{15} of filter to D_{85} of base material does not exceed 4 to 5, base material is prevented from passing through the pores of the filter. Similarly, when the ratio of D_{15} of filter to D_{15} of base material is more than 5 (between 5 to 40), the seepage forces within the filter are controlled up to permissible small magnitudes.

Multilayered filters (generally 3 layers) consisting of materials of increasing permeabilities from the bottom to top are, many a times, provided and are known as **inverted filters.** These filters are costly and should be avoided where possible. The minimum total thickness of filter is 1 m. However, if sufficient quantities of filter material are available at reasonable costs, thicker layers of filter may be provided. The thicker the layer, the greater the permissible deviation from the filter requirements.

20.17. Slope Protection

20.17.1. Protection of Upstream Slope. The upstream slope of the earth dam is protected against the erosive action of waves by stone pitching or by stone dumping, as shown in Fig. 20.34. The thickness of the dumped rock should be about I metre and should be placed over a gravel filter of about 0.3 m thickness. The filter prevents the washing of fines from the dam into the rip-rap. The provision of such a dumped rip-rap has been found to be most effective and has been found to fail only in 5% cases.

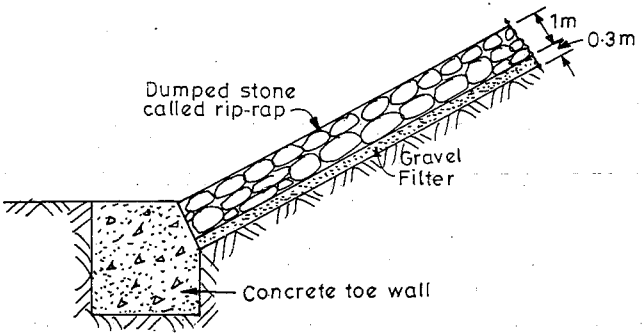


Fig. 20.34

The stone pitching, *i.e.* the hand packed rip rap requires a lesser thickness and may prove more economical if suitable rock is available only in limited quantity. However, when provided in smaller thickness (*i.e.* single layer), it is more susceptible to damage and has been founded to fail in about 30% of cases.

Concrete slabs may also be laid over the u/s slope of the earth dam. When such slabs are constructed, they must be laid over a filter and weep holes should be provided so as to permit escape of water when the reservoir is drawn down. If the filter is not provided, the fines from the embankment may get washed away from the joints creating hollows beneath the slab and causing the consequent cracking and failure of the slab under its own weight. Concrete slab protections have been found to fail in about 36% cases, mainly because of non-providing of filter below them.

20.17.2. Protection of Downstream Slope. The downstream slope of the earthen dam is protected against the erosive action of waves upto and slightly above the water depth, in a similar manner as is explained above for u/s slope.

Moreover, the d/s slope should be protected against the erosive action of rain and its run-off by providing horizontal berms at suitable intervals say about 15 m or so (Fig. 20.5) so as to intercept the rain water and discharge it safely. Attempts should also be made so as to grass and plant the d/s slopes, soon after their construction.

20.18. Rockfill Dams

Rockfill dams have characteristics lying somewhere between the characteristics of gravity dams and those of earthen dams. In other words, they are less flexible than earthen dams and more flexible, then gravity dams. Their foundation requirements are not as strict and rigid as are required for gravity dams. But the foundation requirements

are more rigid than those for earthen dams which can be constructed almost on any types of foundations. The steeper slopes are used in rockfill dams and hence, the base width is quite less. The smaller base width and the possibility of large scale seepage restricts the foundation requirements of such dams.

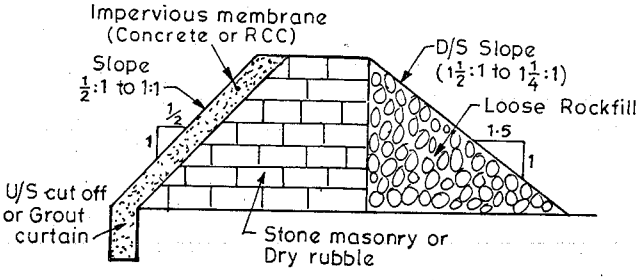


Fig. 20.35. Section of a rockfill dam.

A typical section of a rockfill dams is shown in Fig. 20.35. It essentially consists of an impervious membrane and embankment supporting the membrane. The embankment is divided into two portions. The u/s portion is made of stone masonry or dry rubble, and the d/s portion is made of loose rockfill. The u/s portion embankment supports the membrane and the water load : while the d/s portion supports the u/s embankment, membrane and waterload.

The impervious membrane is usually of concrete, with expansion joints at suitable intervals (say 10 m or so). The expansion joints may be filled with suitable bitumen filler to minimise leakage. Sometimes, R.C.C. membranes are provided with sufficient horizontal and vertical reinforcements, without any expansion joints. The slab thickness is generally less at the top and is more near the dam bottom. The usual slab thickness is between 15 to 50 cm, depending upon the design.

The upstream face containing the membrance, is sloped at a slope varying from say $\frac{1}{2}$: 1 ($\frac{1}{2}H$: 1V) in low dams up to height of about 60 m or so, to 1 : 1 or $1\frac{1}{2}$: 1 in high dams. The d/s slope of all the rockfill dams is kept at about 1.3 : 1, which represents the angle of repose of the rockfill.

Rockfill dams are generally cheaper than concrete dams and can be constructed rapidly if proper rock is available. The rock must be strong enough to withstand high intensity leadings even when wet. The size of loose rock may vary from small stones to 3 m or so. Rockfill dams are very useful in seismic regions, as they provide high resistance to seismic forces because of their flexible character. However, rockfill dams are liable to large settlements, which may lead to cracking of concrete membrane. Repairs for the membrane are, therefore, undertaken from time to time, as and when the necessity arises.

PROBLEMS

1. (*a*) What are 'earthen dams' and under what circumstances are they preferred ?

(*b*) Enumerate the different types of earthen dams, and draw neat sketches showing each type.

[Hint : See article 20.2]

(*c*) Enumerate the two different methods which are adopted for constructing earthen dams. Which of these methods would you prefer and why ? **[Hint** : See article 20.3, preferring rolled fill method]

2. (*a*) What is meant by 'pore water pressure' : and what is its significance in the design of earthen dams?

(*b*) What are 'rockfill dams' and what are their advantages over earthen dams. Draw a neat sketch showing the cross section of a rockfill dam.

3. (*a*) Explain in detail the various forces causing instability in a gravity dam.

(*Madras University, 1975*)

(*b*) Draw a section of an earth dam of 20 m height indicating the various parts of the dam.

(*Madras University, 1975*)

4. (*a*) What are the different types of earth dams that are usually adopted. State where each type is adopted ?

(*b*) What are the causes of failures of earth dam ? (*Madras University, 1974*)

5. What are the precautions that you would take while constructing an earth dam ?

Explain the Swedish slip circle method of analysing the stability of an earth dam slopes.

(*Madras University, 1976*)

6. (*a*) Draw a neat section of an earth dam for the following site :

(*b*) Both silty clay and coarse sand are available at site. Hard stratum is available at about 5 m below the natural ground.

(*c*) Explain briefly how the stability of earthen slopes are checked by the slip circle method.

(*Madras University, 1973*)

7. (*a*) Differentiate between 'Rigid dams' and 'Non-rigid dams' giving examples of each type.

(*b*) Explain with neat sketches how you would carry out the stability analysis of an earth dam.

(*U.P.S.C., 1974*)

8. (*a*) Differentiate between 'horizontal' and 'vertical' piping in earth dams. Suggest permanent measures to check vertical piping.

(*b*) Show with the help of simple outline sketches suitable broad designs of earth dams for different available materials and the governing geology of the site.

Note. At least four different designs should be given.

9. (*a*) Give a suitable design for a 50 m high dam for site where both clay silt and sand gravel are available in plenty and where foundation is pervious to a depth of 10 m. Assume suitable data. Give reasons favouring the suggested design.

10. (*a*) Briefly discuss the checks that are required to be made to investigate the stability of an earthen dam. (*U.P.S.C., 1975*)

(*b*) An earthen dam has to be constructed to store a maximum depth of 12 m of water over river bed consisting of coarse sand and gravel up to a depth of 3 m below river bed followed thereafter by bard and sound rock. Clayee soil is available in plenty in the vicinity of the river. Draw and detail a suitable section of the dam at the river bed.

11. Explain how the following parameters affect design of an earth dam :

(*i*) optimum moisture content ;

(*ii*) C and ϕ value of soil ; permeability of soil

(*iii*) sudden draw-down of the reservoir.

Illustrate with neat sketches the following parts of an earthen dam and state their functions briefly:

(*i*) Rock toe ;

(*ii*) horizontal drainage blanket ;

(*iii*) cut-off ;

(*iv*) Rip-rap.

12. Enumerate and explain by neat sketches the different ways by which the earthen dams may fail. Also suggest suitable precautions that should be undertaken to avoid each type of failure.

13. (*a*) Explain the meaning and importance of equipotential lines' and 'stream lines' in connection with seepage analysis of earthen dams.

(*b*) Derive an expression for calculating the seepage discharge through earthen dam bodies made of isotropic soils. What correction factor is required to be multiplied with this expression, if the dam soil is non-isotropic.

14. (*a*) Define and explain the term 'phreatic line' in earthen dams.

(*b*) How would you procced to determine the phreatic line through homogeneous earthen dams provided :

(*i*) with a horizontal filter ;

(*ii*) Without a horizontal filter.

15. (*a*) Explain and elaborate the importance of 'seepage' through earthen dams.

(*b*) What precautions and remedial measures would you undertake to control the 'seepage' through

(*i*) earthen dam body ;

(*ii*) through the dam foundation.

16. Write short notes on any five of the followings :

(*i*) Rock toe

(*ii*) Climney drain

(*iii*) Relief wells

(*iv*) Slope protection in earthen dams

(*v*) Rock fill dams

(*vi*) Consolidation of earthen dams

(*vii*) Pore pressure and its significance in relation to earthen dam construction

(*viii*) 'Seepage failures' of earthen dams

(*ix*) Rational design of drainage filters for earthen dams.

17. A section of a homogeneous earth dam is shown in Fig. 20.36. Calculate the seepage discharge per metre length, through the body of the dam. The coefficient of permeability of the dam material may be taken as 8×10^{-5} m/sec.

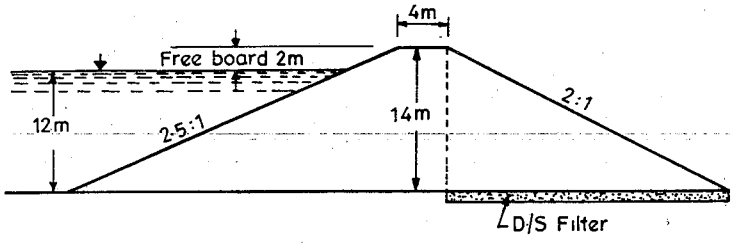


Fig. 20.36

[**Ans.** 2.88×10^{-1} cumecs/m]

18. An earthen dam made of homogeneous material has the following data :

(A) Hydraulic data of dam

Level of top of dam	= 210 m.
Level of deepest river bed	= 192 m.
HFL of reservoir	= 208 m.
Width of top of dam	= 10 m.
Upstream slope	= 3 : 1
Downstream slope	= 2.5 : 1
Length of horizontal filter from d/s toe inwards	= 16 m.

(B) Properties of the material of the dam

Dry density	= 18 kN/m^3
Saturation density	= 21 kN/m^3
Average angle of friction	= 30°
Average cohesion	= 16 kN/m^2.

(C) The foundation soil consists of 4 m thick layer of soil having the following properties :

Average unit weight $= 17\ kN/m^3$

Average cohesion $= 54\ kN/m^2$.

Average angle of internal friction $= 7°$

Check the dam section for the following :

(*i*) Sloughing of u/s slope during sudden drawdown.

(*ii*) Stability of foundation against shear.

[**Hint.** Follow example 20.4]

19. In order to determine the factor of safety of the d/s slope during steady seepage, the section of a homogeneous earth dam was drawn to scale of 1 cm = 10 m ; and the following results were obtained on a trial slip circle.

Area of N-diagram = 12.15 sq cm.

Area of T-diagram = 6.50 sq cm.

Area of U-diagram = 4.02 sq cm.

Length of arc = 11.60 cm.

The dam material has the following properties :

Effective angle of internal friction $= 26°$

Unit of cohesion $= 19.62\ m^2$

Unit weight of soil $= 19.62\ m^2$

Determine the factor of safety of the slope. [**Ans.** F.S. 1.21]

21

Spillways, Energy Dissipators, and Spillway Gates

21.1. Introduction

A spillway is a structure constructed at a dam site, for *effectively disposing of the surplus water from upstream to downstream.* Just after the reservoir gets filled up, up to the normal pool level, water starts flowing over the top of the spillway crest (which is generally kept at normal pool level). Depending upon the inflow rate, water will start rising above the normal pool level, and at the same time, it will be let off over the spillway. The water can rise over the spillway crest, upto the maximum reservoir level, which can be estimated from the inflow flood hydrograph and the spillway characteristics, by the process of flood routing, explained earlier. Therefore, it is only the spillway, which will dispose of the surplus water and will not let the water rise above the maximum reservoir level. Had there been no such structure, over which the water would have overflown, the water level must have exceeded maximum reservoir level, and ultimately would have crossed the freeboard and thus overtopped the dam, causing the failure of the dam. *Hence, a spillway is essentially a safety valve for a dam.* It must be properly designed and must have adequate capacity to dispose of the entire surplus water at the time of the arrival of the worst design flood.

Many dams have failed (especially the earthen dams) because of the improperly designed or inadequate spillways.

21.2. Location of a Spillway

A spillway can be located either within the body of the dam, or at one end of it or entirely away from it, independently in a saddle. If a deep narrow gorge with steep banks, separated from a flank by a hillock with its level above the top of the dam (such as shown in Fig. 21.1), is available, the spillway can be best built independently of the dam.

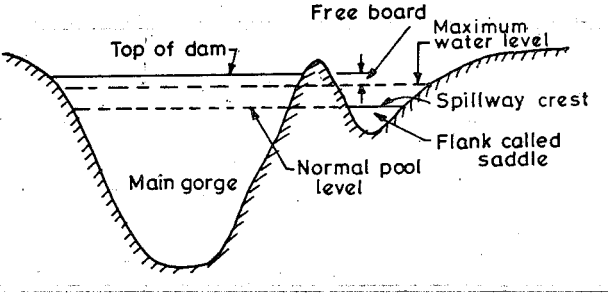


Fig. 21.1

Under such circumstances, a concrete or an earthen dam can be constructed across the main valley and a spillway can be constructed independently into the saddle. Sometimes, a concrete or a masonry dam along with its spillway can be constructed in the main valley, while the flank or flanks are closed by earthen dikes or embankments. The

top level of such an embankment is kept at maximum reservoir level. The materials and designs of these embankments are such that they fail as soon as water overtops them. Hence, if by chance, either due to excessive flood above the design flood or due to failure of gates of main spillway, etc., the water rises above the maximum reservoir level, it shall overtop such embankment, which at once fails ; providing sufficient outlet for the disposal of excessive water. This type of a secondary safety arrangement is generally provided on large dams especially on earth and rockfill dams, and is *known as Subsidiary Spillway* or *Emergency Spillway* or *Breaching Section.*

The main spillway is constructed to dispose of the designed flood above the normal pool level and upto the maximum reservoir level. It is situated either within the dam, or at one end of it, or independently in a saddle away from the main dam. A separate independent spillway is generally preferred for earthen dams, although due to non-availability of sites, a concrete spillway is sometimes constructed within or at one of the ends of an earth dam. If the main spillway is situated in a flank, a secondary emergency spillway may be situated in another flank as shown in Fig. 21.2.

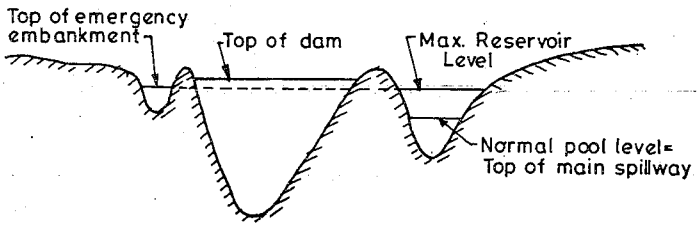


Fig. 21.2

21.3. Design Considerations for the Main Spillway

The main spillway, often called the spillway, is properly designed so as to dispose of the excess water without causing any damage to the dam, or to any of its appurtenant structures. The spillway structure should be structurally and hydraulically adequate and must not give way under worst and variable loading conditions.

The required discharging capacity of the spillway should be as closely estimated as possible. The underestimation will lead to overtopping of the main dam and its consequent damages ; while the over estimation will lead to unnecessarily costly constructions which shall never he utilised during the life of the dam, and hence, will remain a waste investment. However, on large dams, a conservative view is always preferred because the failure of a single dam due to inadequate capacity may result in the loss of numerous human lives to which no cost allocation can be made. Moreover, an emergency spillway or a breaching section is generally provided, the failure of which under necessary circumstances, may though cause serious erosion on the downstream, but shall protect the main dam from failure.

The water passing over the spillway and falling on the downstream side must not be allowed to erode the downstream soil, and hence, arrangements must be made for effectively dissipating the energy of the falling water.

21.4. Controlled and Uncontrolled Spillways

The flow of water over a spillway may be controlled by installing gates over the spillway crest. In such a case, the spillway is known as a controlled spillway. The out flow can be controlled in such spillways and hence, preferred in modern days. However, some spillways are left just by constructing their crest at normal pool level. As water

will flow over such a spillway, depending upon the reservoir level and the corresponding head over the spillway, such uncontrolled spillways are guided only by the available water head, and hence, are called uncontrolled spillways. The comparative advantages and disadvantages of controlled and uncontrolled reservoirs have been discussed in chapter 18.

VARIOUS TYPES OF SPILLWAYS

Depending upon the type of the structure constructed for disposing of the surplus water, the spillways can be of the following major types :

(1) Straight Drop Spillway.

(2) Overflow Spillway generally called *Ogee Spillway.*

(3) *Chute Spillway* often called *Trough Spillway* or Open channel Spillway.

(4) Side Channel Spillway.

(5) Shaft Spillway.

(6) Syphon Spillway.

The various types of spillways enumerated above are described below along with the design details of 'Ogee Spillway' and 'Chute Spillway'.

21.5. Straight Drop Spillway or Overfall Spillway

This is the simplest type of spillway and may be constructed on small bunds or on thin arch dams, etc. It is a low weir and simple vertical fall type structure, as shown in Fig. 21.3. The downstream face of the structure may be kept vertical or slightly inclined. The crest is sometimes extended in the form of an overhanging lip, which keeps small discharges away from the face of the overfall section. The water falls freely from the crest under the action of gravity. Since vacuum gets created in the underside portion of the falling jet, sufficient ventilation of the nappe is required in order to avoid pulsating and fluctuating effects of the jet. The design of such a spillway is done as that of a weir which was explained in the chapter on

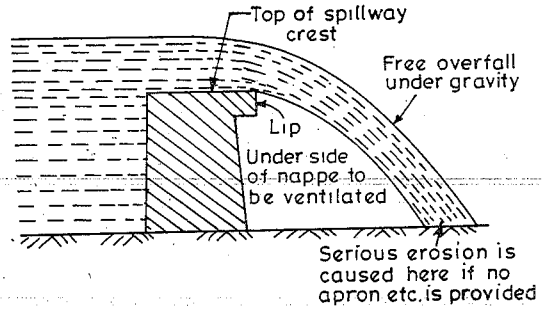


Fig. 21.3. (*a*) Straight drop spillway without d/s protection.

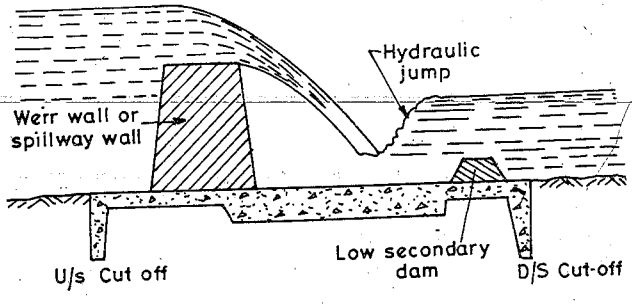


Fig. 21.3. (*b*) Straight drop spillway with d/s protection works.

weirs. Sometimes, a secondary dam of low height is constructed on the downstream side to create an artificial pool of water so as to dissipate the energy of the falling water.

21.6. Ogee Spillway or Overflow Spillway

Ogee spillway is an improvement upon the 'free overfall spillway, and is widely used with concrete, masonry, arch and buttress dams. Such a spillway can be easily used on valleys where the width of the river is sufficient to provide the required crest length

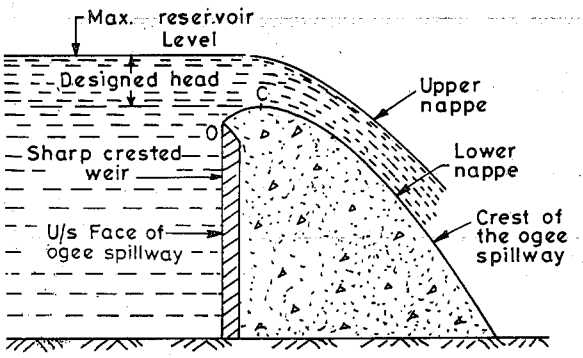


Fig. 21.4. (*a*) Section of an ogee spillway with vertical u/s face.

and the river bed below can be protected from scour at moderate costs. *The profile of this spillway is made in accordance with the shape of the lower nappe of a free falling jet, over a duely ventilated sharp crested weir,* as shown in Fig. 21.4 (*a*).

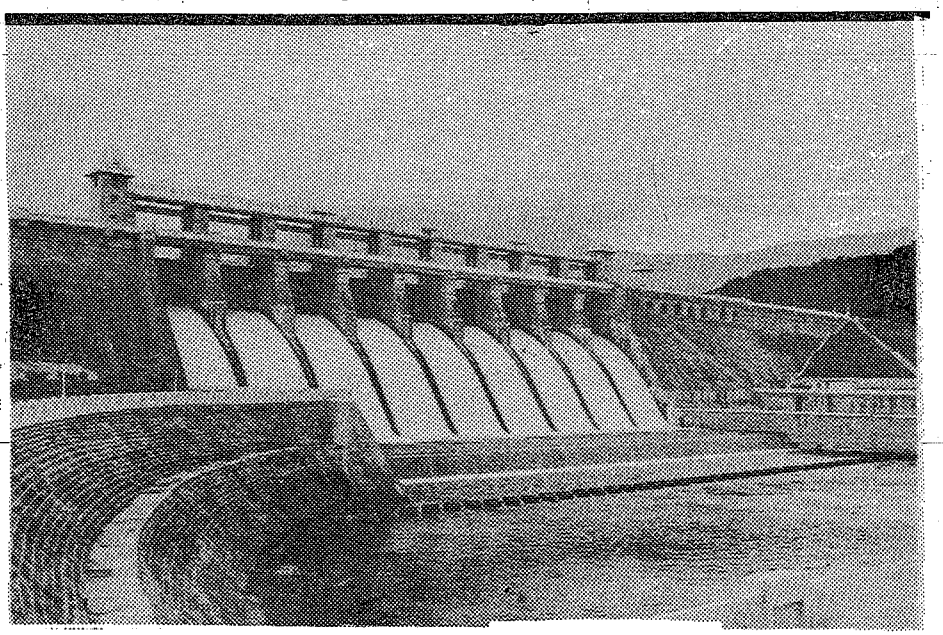


Fig. 21.4. (*b*) Photoview of an Ogee Spillway of Amaravathi dam (earthern cum masonry dam) located on river Amaravathi (a tributary of river Cauvery) in Coimbatore District in Tamil Nadu state.

The shape of the lower nappe of freely falling jet over a sharp crested weir can be determined by the principle of projectile. It generally rises slightly (to point *C*) as it originates from the crest (*O*) of a sharp crested weir and then falls to make a parabolic form. Now, if the space between the sharp crested weir and the lower nappe is filled with concrete or masonry, the weir so formed will have a profile similar to an 'ogee' (*S*-shaped curve in section), and hence called an 'ogee weir' or an 'ogee spillway'. This lower nappe, will then become the crest of the spillway. Since the lower nappe of the free falling jet will be different for different heads over the crest of the sharp crested weir, *the profile of the ogee weir is generally confined to the lower nappe that would be obtained for maximum head over the spillway (i.e. upto the maximum reservoir level).*

In a free overfall spillway, the water jet falls clearly away from the face of the spillway, and the gap between the jet and the face is kept ventilated. While in an ogee spillway, the falling water glides over the curved profile of the spillway, and there is no space between water and crest of the spillway, under normal design conditions.

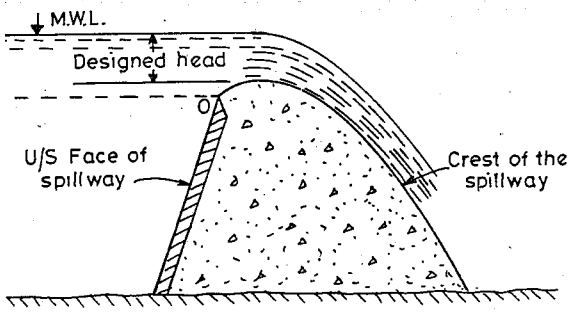


Fig. 21.5. Ogee spillway with inclined u/s face.

Normally, the upstream face of the spillway is kept vertical and the crest shape confirms to the lower nappe of a vertical sharp crested weir under maximum head. But if the upstream face of the spillway is kept sloping, the crest shape should also confirm to the lower nappe that would be obtained for an inclined sharp crested weir (Fig. 21.5).

21.6.1. Cavitation. The crest of the ogee spillway can be made to confirm only to one particular nappe that would be obtained at one particular head. This head is called the *designed head* and represented by, say, H_d. But in practice, the actual head of water on the spillway crest, called the *operating head,* may be less or more than the designed head. *If this operating head on the spillway is more than the designed head, the lower nappe of the falling jet may leave the ogee profile, thereby generating negative pressure at the point of separation.* The generation of vacuum or negative pressure (*i.e.* pressure below the atmospheric pressure) may lead to formation of bubbles or cavities in the water. These cavities or bubbles filled with air, vapour and other gases are formed in a liquid, whenever the absolute pressure (*i.e.* atmospheric pressure—vacuum pressure) of the liquid is close to its vapour pressure, so as to commence evaporation. Such a condition may arise when the head of water is more than the designed head and the consequent high velocity jet causes reduced pressures or negative pressures in the lower region of the water jet.

Such cavities, on moving downstream, may enter a region where the absolute pressure is much higher (*i.e.* more vacuum). This causes the vapour in the cavity to condense and return to liquid with a resulting implosion or collapse of the cavity. When

the cavity collapses, extremely high pressures are generated. The continuous bombardment of these implosions will thus take place near the surface of the spillway, causing fatigue failure of its material. The small particles of concrete or masonry are thus broken away, causing formation of pits on its surface and giving the surface a spongy appearance. This damaging action of cavitation is called 'pitting'. The cavitation plus the vibrations from the alternate making and breaking of contact between the water and face of the spillway, may thus result in serious structural damages to the spillway crest. *Hence, it can be concluded that if the head of water over the spillway is more than the designed head, cavitation may occur. On the other hand, if the head of water over the spillway is less than the designed head, the falling jet would adhere to the crest of the ogee spillway, creating positive hydrostatic pressures and thereby reducing the discharge coefficient of the weir.*

21.6.2. Designing the Crest of the Ogee Spillway. The ogee spillways were being designed in the earlier periods, in accordance with the theoretical profile obtained for the lower nappe of a free falling jet. The profile was known as Bazin's profile. Theoretically, the adoption of such a profile, should cause no negative pressures on the crest under designed head. But in practice, there exists a lot of friction due to roughness on the surface of the spillway. Hence, negative pressure on such a profile seems inevitable. The presence of negative pressure causes the danger of cavitation and sometimes fluctuations and pulsations of the nappe. Hence, while adopting a profile for the spillway crest, the avoidance of negative pressures must be an objective along with consideration of other factors such as practicability, hydraulic efficiency, stability and economy. Depending upon research work based on these objectives, various modified profiles have been proposed these days.

Several standard ogee shapes have been developed by U.S. Army Corps of Engineers at their Waterways Experimental Station (WES). Such shapes are known as 'WES Standard Spillway Shapes'. The d/s profile can be represented by the equation

$$x^n = K \cdot H_d^{n-1} \cdot y \qquad ...(21.1)$$

where (x, y) are the co-ordinates of the points on the crest profile with the origin at the highest point C of the crest, called the apex.

H_d is the *design head including the velocity head.*

K and n are constants depending upon the slope of the upstream face. The values of K and n are tabulated in Table 21.1.

Table 21.1

Slope of the u/s face of the spillway	*K*	*n*
Vertical	2.0	1.85
1 : 3 (1*H* : 3*V*)	1.936	1.836
1 : $1\frac{1}{2}$ (1*H* : $1\frac{1}{2}$*V*)	1.939	1.810

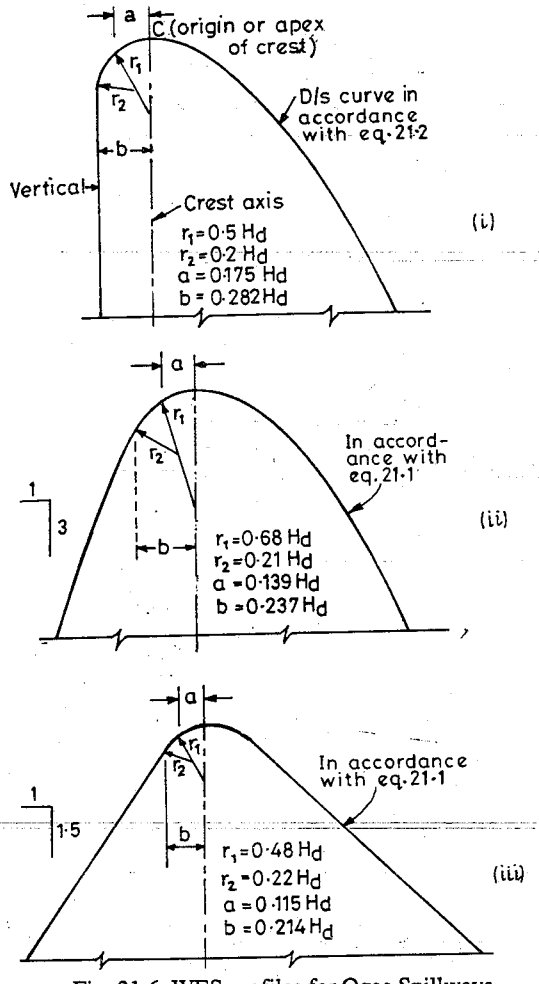


Fig. 21.6. WES profiles for Ogee Spillways for different u/s slopes.

Thus, for a spillway having a vertical u/s face, the d/s crest is given by the equation

$$x^{1.85} = 2 \cdot H_d^{0.85} \cdot y \qquad ...(21.2)$$

Different upstream curves were given by WES for different slopes, as shown in Fig. 21.6.

According to the latest studies of U.S. Army Corps, the u/s curve of the ogee spillway having a vertical u/s face, should have the following equation :

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375} \times (x + 0.27\,H_d)^{0.625} \qquad ...(21.3)$$

The u/s profile extends up to

$$x = -0.27\,H_d$$

Co-ordinates for the upper nappe for various WES shapes of ogee spillway are also available and can be utilised in the design of training walls and spillway bridges, etc.

The profile for an ogee spillway, having a vertical upstream face, can be determined on the basis of its WES profile, or Table 21.2 may be used for making the ogee profile.

Table 21.2. Table for marking Ogee profile

$\frac{x'}{H_d}$	$\frac{y}{H_d}$	
	Lower Nappe	*Upper Nappe*
0	– 0.125	0.831
0.10	– 0.033	0.807
0.25	– 0.000	0.763
0.50	– 0.034	0.668
0.75	– 0.129	0.539
1.0	– 0.283	0.373
1.5	– 0.738	– 0.088
2.0	– 1.393	– 0.743
3.0	– 3.303	– 2.653
4.0	– 6.013	– 5.363
5.0	– 9.523	– 8.873

(x', y) are the co-ordinates of a point on the profile as shown in Fig. 21.7.

The co-ordinates of the lower nappe determine the crest profile, while the plotting of the upper nappe is useful in determining the clearance for the spillway deck bridge and the top levels of training walls on the sides of the spillway.

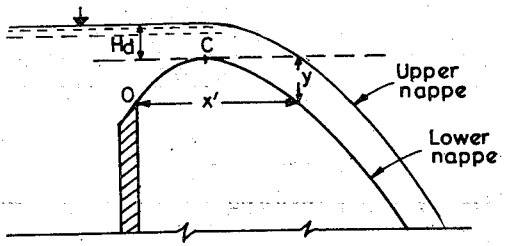


Fig. 21.7

After having plotted most of the profile of the ogee spillway, a smooth gradual reverse curvature is provided at the bottom of the downstream face, as shown in Fig. 21.8. The reverse curve turns the flow into the apron of a stilling basin or into the spillway discharge channel. A radius of about one-fourth of the spillway height is satisfactory for this reverse bottom curve.

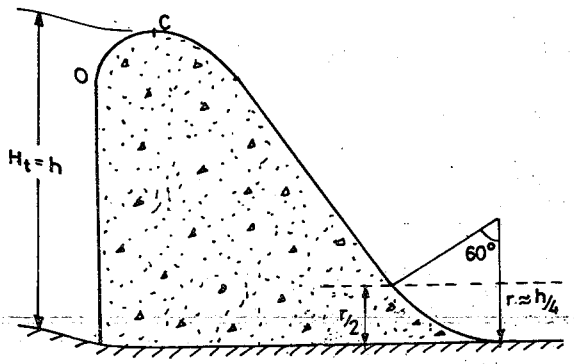


Fig. 21.8

21.6.3. Practical Profile Consistent with the Profile of Gravity Dam. When the profile for the crest of the ogee spillway is plotted over the triangular profile of the section of a gravity dam (non-overflow section), it is found that it goes beyond the downstream face of the dam, thus requiring thickening of the section for the spillway (Fig. 21.9). This extra concrete required, can be saved by shifting the curve of the nappe in a backward direction until this curve becomes tangential to the downstream face of the dam (Fig. 21.10). Hence, a saving can thus be affected, by providing a projecting corbel on the upstream face of the spillway section (Fig. 21.10). The construction of the spillway is thus carried out as if it was a non-overflow dam. Only the slight modifications are made after reaching the required height (up to O) at

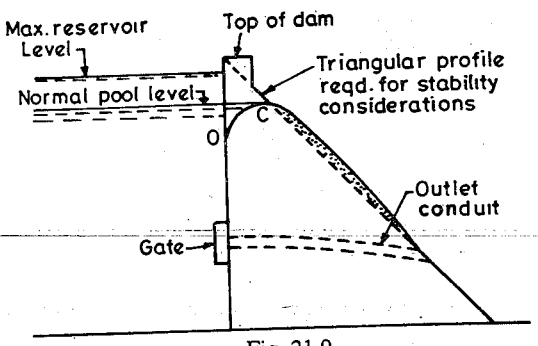


Fig. 21.9

which corbel is provided, and a smooth required curve *OCA* is given, as shown in Fig. 21.10.

If the spillway is provided with gates on the upstream face to control the flow in outlet conduits (Fig. 21.9), then the corbel will interfere with gate operation and hence the above concrete saving cannot be affected.

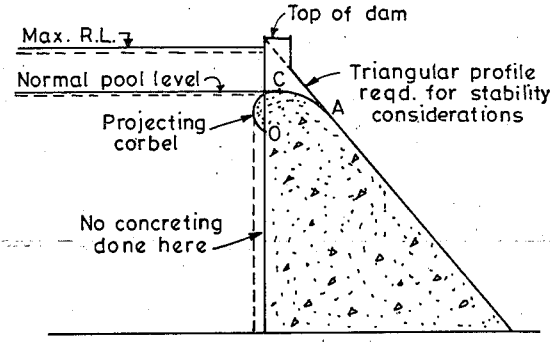


Fig. 21.10. Provision of Corbel.

The structural design and stability requirements of ogee spillway are exactly the same as that of a gravity dam. The forces acting on a gravity dam also act on the ogee spillway and remain predominant. The pressure exerted on the crest of the spillway by the flowing water and the drag force caused by the fluid-friction are usually negligible as compared to other forces which are acting on a gravity dam and also act on an ogee spillway. Hence, the design and construction of the ogee spillway is consistent with that of the gravity dam. Due to this reason, the spillway is sometimes called overflow portion of the dam, and the real dam section is termed as non-overflow section of the dam.

However, the change in momentum of the flow in the vicinity of the reverse curve may create a dynamic force on the spillway, in addition to the forces acting on the gravity dam. This force must be considered in designing bottom of the d/s face of spillway or the bucket type energy dissipator. This is dealt separately while discussing the design of energy dissipators for the spillways.

21.6.4. Discharge Formula for the Ogee Spillway. The discharge passing over the ogee spillway is given by the equation :

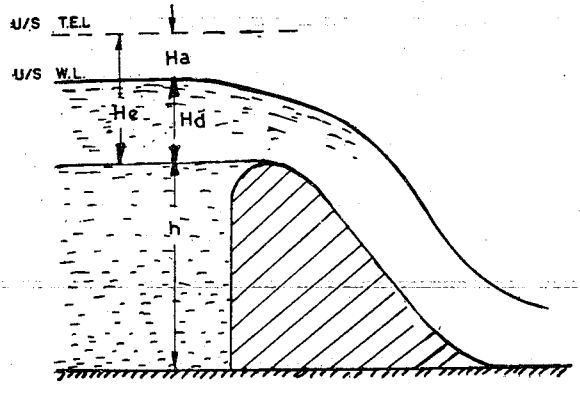


Fig. 21.11

$$Q = C \cdot L_e H_e^{3/2} \qquad ...(21.4)$$

where Q = Discharge

L_e = Effective length of the spillway crest

C = Coefficient of discharge which depends upon various factors such as relative depth of approach, [*i.e.* d/H_d ratio (Fig. 21.11), relation of actual crest shape to the ideal nappe shape, slope of upstream face, downstream apron interference, and submergence, etc.

H_e = Total head over the crest including the velocity head.

If the discharge Q is used as the design discharge in Equation (21.4), then the term H_e will be the corresponding design head (H_d) plus the velocity head (H_a). In such a case, $H_e = H_d + H_a$. For high ogee spillways, the velocity head is very small, and $H_e \approx H_d$.

21.6.5. Variation of Coefficient of Discharge with Various Factors. This is discussed below in details :

(*i*) *Depth of approach.* The coefficient of discharge firstly depends upon the depth of approach. In other words, it depends upon the height of the ogee weir (h) to the design head over the weir (H_d). If the height of the weir is more than 1.33 times the design head, the velocity of approach has been found to have a negligible effect upon discharge, and as such H_d becomes equal to H_e or $\frac{H_e}{H_d} = 1.0$. In such a case, the coefficient of discharge, say $C = C_d$, has been found to *be 2.2 in M.K.S. or S.I. Units* and 4.03 in *F.P.S. Units.*

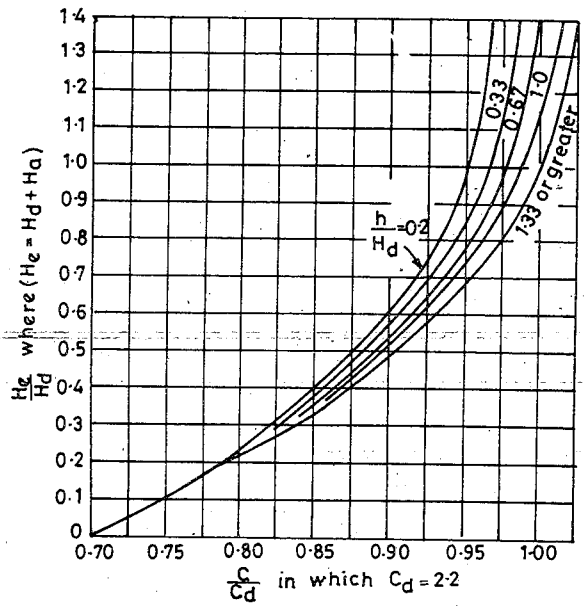


Fig. 21.12

However, in low spillways, with $\frac{h}{H_d} < 1.33$; the approach velocity is having an appreciable effect. The curves given in Fig. 21.12 can be used in such cases, to evaluate the coefficient of discharge C, using $C_d = 2.2$.

(*ii*) *U/s Slope.* The coefficient of discharge is also affected by the slope of the u/s face of the ogee weir. The values of C_d and C found up to now were for a vertical upstream face. If the u/s face is sloping, a correction factor by which the above values of C should be multiplied can be obtained from the curves given in Fig. 21.13.

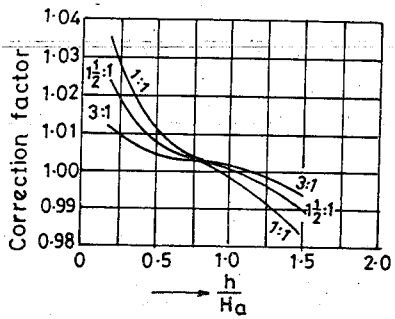


Fig. 21.13

(iii) D/s apron interference and submergence effects. The third important factor affecting the discharge coefficient of an ogee weir is the effect of downstream apron interference and downstream submergence.

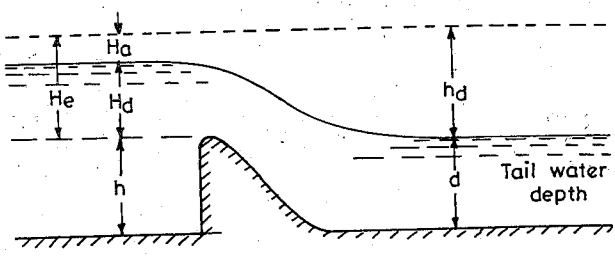


Fig. 21.14. Maximum tail water depth for a non-submerged weir.

When the tail water level is such that the top of the weir is covered by it, such that the weir cannot discharge freely ; the weir is then said to be a **submerged weir.**

Where the hydraulic jump occurs, the coefficient of discharge may decrease due to back pressure effect of the downstream apron and is independent of the submergence effect. When the value of $\frac{h_d + d}{H_e}$ (Fig. 21.15) exceeds 1.7, the downstream apron is found to have negligible effect on the coefficient of discharge. But there may be a decrease in the coefficient due to tail water submergence. The correction factor, by which the value of C should be multiplied in order to get the modified or correct value of coefficient of discharge, can be obtained from the curve of Fig. 21.15.

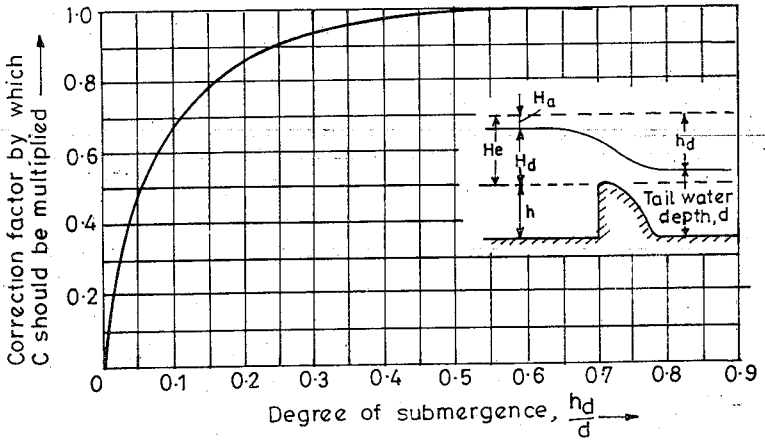


Fig. 21.15. Effect of submergence on C.

21.6.6. Effects of Actual Prevailing Head on the Discharge Capacity of a Spillway. It was pointed out earlier that when once a spillway has been designed and constructed for a design head (H_d), and for a corresponding coefficient of discharge (say C), it will not always find the same head over its crest in its actual operations. The actual operating head (H) *i/c* velocity head, may be less or more than the designed head. Since the design is done for maximum head, the possibility of a head more than the designed head is very meagre. When the actual operating head passing over the spillway is less than the designed head, the prevailing coefficient of discharge (C_d) tends to reduce, and is given by the equation

$$C_d = C \cdot \left(\frac{H}{H_e}\right)^{0.12} \qquad ...(21.5)$$

where H_e is the designed head including velocity head.

Since an overflow spillway is sufficient in height (*i.e* $h > 1.33\ H_d$) ; the coefficient of discharge C at designed head can be taken as 2.2. The prevailing coefficient of discharge at 50% head (*i/c* velocity head) will then be

$$C_d = 2.2 \times \left(\frac{0.5\ H_e}{H_e}\right)^{0.12} = 2.2 \times 0.92 = 2.02.$$

Similarly, for still lower heads, the coefficient of discharge goes on reducing and tends to become constant at about 1.7. (Because at very low heads, the velocity head becomes the governing factor, which tries to make H a constant).

The effective length (L_e) of ogee spillway.

The effective length of the spillway crest (L_e) to be used in equation (21.4), is given by the equation

$$L_e = L - 2\left[K_p \cdot N + K_a\right] H_e \qquad ...(21.6)$$

where L = the net clear length of the spillway crest

K_p = Pier constraction coefficient

K_a = Abutment contraction coefficient

N = Number of piers

H_e = Total design head on the crest including velocity head.

The values of K_p and K_a depend mainly upon the shape of the piers and that of the abutments. The greater is the divergence from streamlined flow, the greater is the constraction coefficient and lesser is the effective length of the crest. A 90° cut water nose pier is most efficient and has quite a low value of K_p, and is generally preferred. Values of K_p and K_a are given in Tables 21.3 and 21.4. Various shapes of piers are shown in Fig. 21.16.

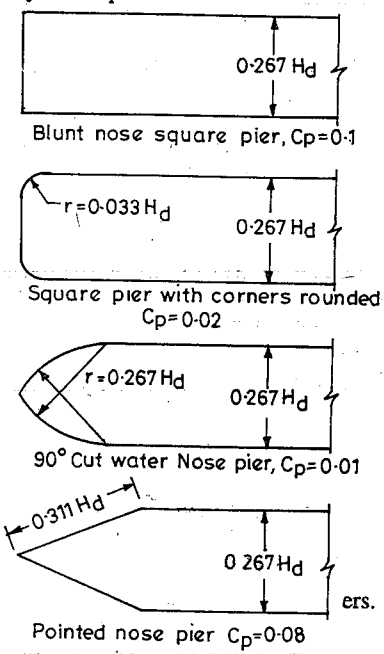


Fig. 21.16. Various shapes of piers.

Table 21.3

S. No.	Pier Shape	Contraction coefficient K_p
1.	Square nosed piers without any rounding	0.1
2.	Square nosed piers with corners rounded on radius equal to 0.1 of pier thickness	0.02
3.	Rounded nose piers and 90° cut water nosed piers	0.01
4.	Pointed nose piers	0.0

Table 21.4

S. No.	*Shape of abutment*	*Contraction coefficient* K_a
1.	Square abutment with head wall at 90° to the direction of flow	0.2
2.	Rounded abutment with head wall at 90° to the direction of flow	0.1

21.6.7. Aeration Arrangements in Gated Ogee Spillways. The entire design of an overflow spillway has been done with the assumption that upper and lower nappe are subjected to full atmospheric pressure. But in practice, due to insufficient aeration, development of negative pressures takes place beneath the nappe due to removal of air by the falling jet. The development of negative pressures causes the danger of cavitation and induces fluctuation and pulsation effects on the jet, which may be very objectionable if the spillway is to be used as a discharge measuring device.

To control the development of negative pressures, aeration pipes 25 mm dia at say 3 m centre to centre may be provided along spillway face below the gate lip. These pipes can be connected to a bigger sized header. Formulas have been developed on the basis of model studies, for evaluating the amount of air required for aeration, and may have to be used in large dam designs, but are beyond the scope of this book.

Example 21.1. *Design a suitable section for the overflow portion of a concrete gravity dam having the downstream face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 8,000 cumecs. The height of the spillway crest is kept at RL 204.0 m. The average river bed level at the site is 100.0 m. The spillway length consists of 6 spans having a clear width of 10 m each. Thickness of each pier may be taken to be 2.5 m.*

Solution. Since the given spillway looks like a high weir, the coefficient of discharge may be assumed to be 2.2.

Now $$Q = C \cdot L_e H_e^{3/2}$$

where $L_e = L - 2\,[N K_p + K_a]\, H_e$

Let us first work out the approximate value of H_e for a value of

$$L_e \approx L = \text{clear waterway} = 6 \times 10 = 60 \text{ m}.$$

$$\therefore \quad 8{,}000 = 2.2 \times 60\, H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 60} = 60.6$$

or $$H_e = (60.6)^{2/3} = 15.5 \text{ m}.$$

The height of the spillway above the river bed (see Fig. 21.15)

$$= h = 204 - 100 = 104.0 \text{ m}$$

Since $\frac{h}{H_d}$, *i.e.* $\frac{104}{15.5} > 1.33$,

it is a high spillway, the effect of velocity head can, therefore, be neglected.

Since $$\frac{h_d + d}{H_e} = \frac{H_e + h}{H_e} = \frac{15.5 + 104}{15.5} > 1.7;$$

the discharge coefficient is not affected by tail water conditions, and the spillway remains a high spillway.

U/s Slope. The upstream face of the dam and spillway is proposed to be kept vertical. However, a batter of 1 : 10 will be provided from stability considerations in the lower part. This batter is small and will not have any effect on the coefficient of discharge.

Effective length of spillway (L_e) can now be worked out as

$$L_e = L - 2\,[N.K_p + K_a]\,H_e$$

Assuming that 90° cut water nose piers and rounded abutments shall be provided, we have

$$K_p = 0.01$$

and $$K_a = 0.1$$

No. of piers $= N = 5$.

Also assuming that the actual value of H_e is slightly more than the approximate value worked out (*i.e.* 15.5 m), say, let it be 16.3 m, we have

$$\therefore \quad L_e = 60 - 2\,[5 \times 0.01 + 0.1] \times 16.3 = \mathbf{55.1\ m.}$$

Hence $$Q = 2.2 \times 55.1 \times H_e^{3/2}$$

or $$8{,}000 = 2.2 \times 55.1 \times H_e^{3/2}$$

or $$H_e^{3/2} = \frac{8{,}000}{2.2 \times 55.1} \cong 66.0$$

or $$H_e = (66.0)^{2/3} = \mathbf{16.4\ m} \cong 16.3 \text{ (assumed)}$$

Hence, the assumed H_e for calculating L_e is all right. The crest profile will be designed for $H_d = 16.4$ m (neglecting velocity head).

Note. The velocity head (H_a) can also be calculated as follows :

$$\text{Velocity of approach} = V_a = \frac{8{,}000}{(60 + 5 \times 2.5)\,(104 + 16.4)}$$

$$= \frac{8{,}000}{72.5 \times 120.4} = 0.917 \text{ m/sec.}$$

$$H_a = \text{Velocity Head} = \frac{V_a^2}{2g} = \frac{(0.917)^2}{2 \times 9.81} = 0.043 \text{ m.}$$

This is very small and was, therefore, neglected.

Downstream profile. The W.E.S. d/s profile for a vertical u/s face is given by equation (21.2) as :

$$x^{1.85} = 2 \cdot H_d^{0.35} \cdot y$$

or $$y = \frac{x^{1.85}}{2\,(H_d)^{0.85}} = \frac{x^{1.85}}{2 \times (1.64)^{0.85}}$$

or $$y = \frac{x^{1.85}}{2 \times 10.8}$$

or $$y = \frac{x^{1.85}}{21.6} \quad ...(21.7)$$

Before we determine the various co-ordinates of the d/s profile, we shall first determine the tangent point.

The d/s slope of the dam is given to be 0.7 H : 1 V.

Hence, $$\frac{dy}{dx} = \frac{1}{0.7}$$

Differentiating the equation of the d/s profile w.r. to x, we get

$$\frac{dy}{dx} = \frac{1.85x^{1.85-1}}{21.6} = \frac{1}{0.7}$$

or $$x^{0.85} = \frac{21.6}{1.85 \times 0.7} = 16.7$$

or $$x = 22.4 \text{ m.}$$

$\therefore$ $$y = \frac{(22.4)^{1.85}}{21.6} = 14.6 \text{ m.}$$

The co-ordinates from $x = 0$ to $x = 22.4$ m are worked out in Table 21.5.

Table 21.5

x *metres*	$y = \frac{x^{1.85}}{21.6}$ *metres*
1	0.046
2	0.166
3	0.354
4	0.60
5	0.905
6	1.274
7	1.710
8	2.162
9	2.684
10	3.240
12	4.575
14	6.020
16	7.88
18	9.74
20	11.85
22	14.35
22.4	14.60

The u/s profile. The u/s profile may be designed as per equation (21.3), as :

$$y = \frac{0.724\,(x + 0.27\,H_d)^{1.85}}{H_d^{0.85}} + 0.126\,H_d - 0.4315\,H_d^{0.375}\,(x + 0.27\,H_d)^{0.625}$$

Using $H_d = 16.4$ m, we get

$$y = \frac{0.724\,[x + 0.27 \times 16.4]^{1.85}}{(16.4)^{0.85}} + 0.126\,(16.4) - 0.4315\,(16.4)^{0.375}\,(x + 0.27 \times 16.4)^{0.625}$$

or $$y = 0.07\,(x + 4.44)^{1.85} + 2.07 - 1.234\,(x + 4.432)^{0.625} \quad ...(21.8)$$

This curve should go upto $x = -0.27\,H_d$

or $$x = -0.27 \times 16.4 = -4.443 \text{ m.}$$

Various values of x such as, $x = -0.5, x = -1.0, x = -2.0, x = -3.0, x = -4.0, x = -4.443$ are substituted in equation (21.8) and corresponding values of y are worked out, as given below in Table 21.6.

Table 21.6

x in metres	*y in metres*
– 0.5	0.020
– 1.0	0.063
– 2.0	0.27
– 3.0	0.65
– 4.0	1.34
– 4.443	2.07

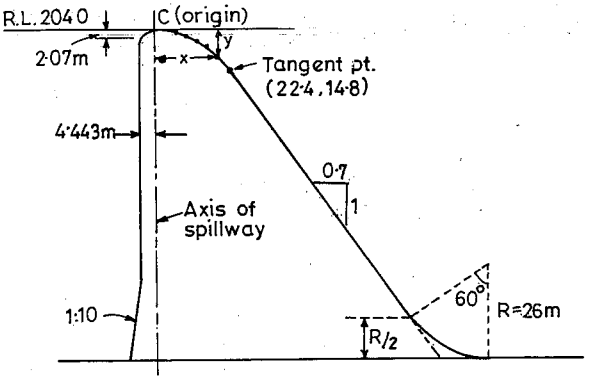


Fig. 21.17

The profile of the spillway has been determined and plotted in Fig. 21.17. A reverse curve at the toe with a radius equal to $\frac{h}{4} = \frac{104}{4} = 26$ m can be drawn at angle 60°, as shown in Fig. 21.17. Aeration pipes (say 25 mm pipes at 3 m c/c) can be installed along the spillway face below the gate lip, so as to prevent the development of negative pressures. The energy dissipation arrangements have not been shown. They should be designed depending upon the position of the jump height curve and tail water curve, as explained afterwards. A sky jump bucket or an apron may be provided as per the prevailing conditions.

21.7. Chute Spillway or the Trough Spillway

An ogee spillway is mostly suitable for concrete gravity dams especially when the spillway is located within the dam body in the same valley. But for earthen and rockfill dams, a separate spillway is generally constructed in a flank or a saddle, away from the main valley, as explained earlier. Sometimes, even for gravity dams, a separate spillway is required because of the narrowness of the main valley. In all such circumstances, a separate spillway may have to be provided. The *Trough Spillway* or *Chute Spillway* is the simplest type of a spillway which can be easily provided independently and at low costs. It is lighter and adaptable to any type of foundations ; and hence provided easily on earth and rockfill dams. A chute spillway is sometimes known as a **waste weir**. If it

is constructed in continuation to the dam at one end, it may be called a **flank weir**. If it is constructed in a natural saddle in a bank of the river separated from the main dam by a high ridge, it is called a **saddle weir.**

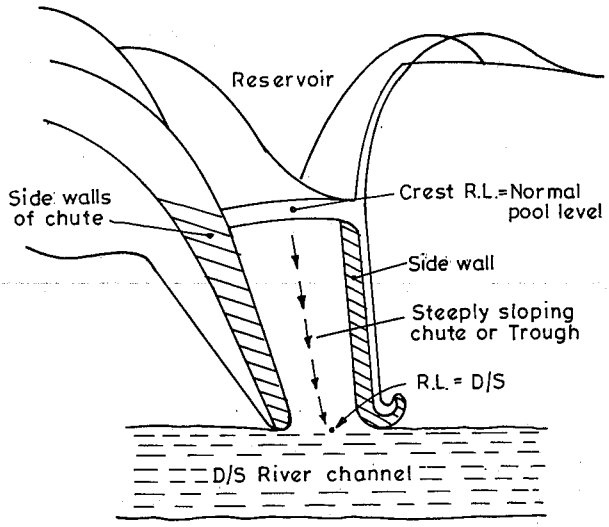


Fig. 21.18 (*a*). Simplified Line sketch of a Chute Spillway.

A chute spillway (Fig. 21.18) essentially consists of a steeply sloping open channel, placed along a dam abutment or through a flank or a saddle. It leads the water from the reservoir to the downstream channel below. The base for the channel is usually made of reinforced concrete slabs, 25 to 50 cm thick. Light reinforcement of about 0.25% of the concrete area is provided in the top of the slabs in both directions. The chute is sometimes of constant width, but is usually narrowed for economy and then widened near the end to reduce the discharging velocity. Expansion joints are usually provided in the chute at intervals of about 10 metres in either direction. The expansion joints should be made watertight so as to avoid any under-seepage and its troublesome effects. Under-drains are also provided, so as to drain the water which may seep through the trough bottom and side walls. These under-drains may be in the form of perforated steel pipes, clay tiles or rock-filled trenches.

If the slope of the chute can confirm to available topography, the excavations shall be minimum. But the slope of the chute must be high enough, and should atleast be able to maintain super critical flow, to avoid unstable flow conditions.

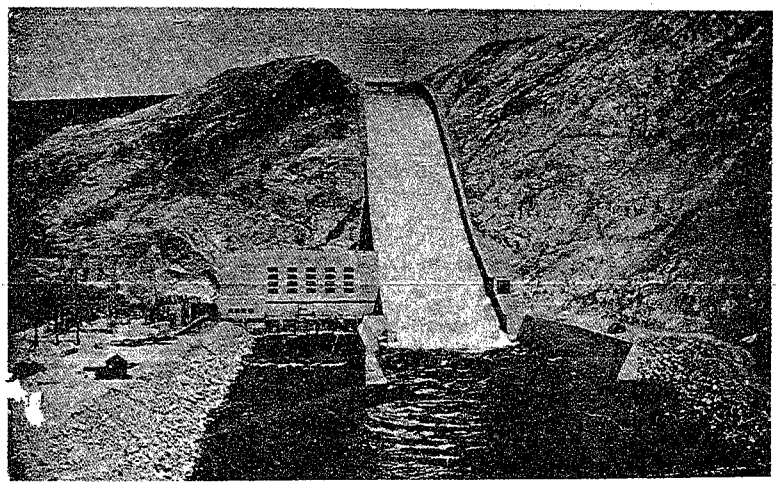


Fig. 21.8. (*b*) Photoview of the Chute Spillway of Anderson Ranch Dam, Idaho (USA).

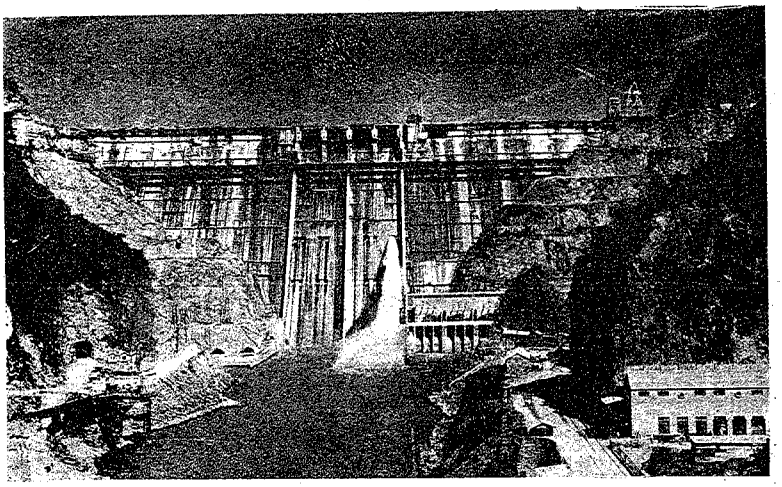


Fig. 21.18. (*c*) Photoview of two chutes provided at Bhakra Dam (India).

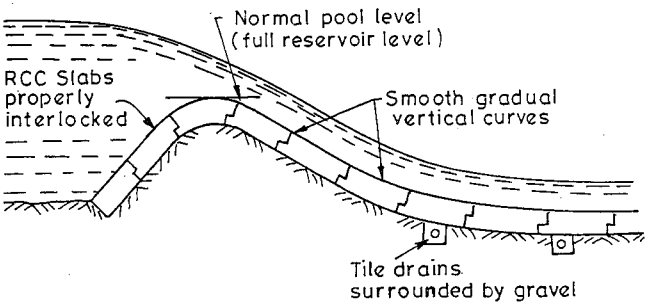


Fig. 21.18. (*d*) Section through a Chute Spillway.

When a vertical curve is provided at a point where the chute slope changes, it must be gradual and designed so as to avoid any separation of flow.

21.7.1. Control Structure or a Low Ogee Weir. Since the trough spillway is provided in a flank or a saddle, the height of spillway or ogee weir required to be constructed in that flank, will be small ; sometimes almost flat low weir shall be required depending on the natural levels of the bottom of the flank. If the flank bottom is at a level lower than the natural pool level, an ogee weir shall have to be constructed upto that level. If the flank bottom is at higher level than the normal pool level, excavations will have to be done upto that level. In such a case, the weir crest is normally left flat as it shall seldom be economical to excavate the rock just for the sake of constructing on 'ogee shape' for obtaining high coefficient of discharge.

21.7.2. Chute Slope. The water spilling over the control structure (*i.e.* ogee weir), then flows through the chute channel. The minimum slope of the chute is governed by the condition that supercritical flow must be maintained. The slope of the chute is kept just sufficient to meet this flow requirement from the crest for as long a distance as possible without any filling. After that, the slope is made as steep as possible without endangering the stability or without getting into heavy excavations.

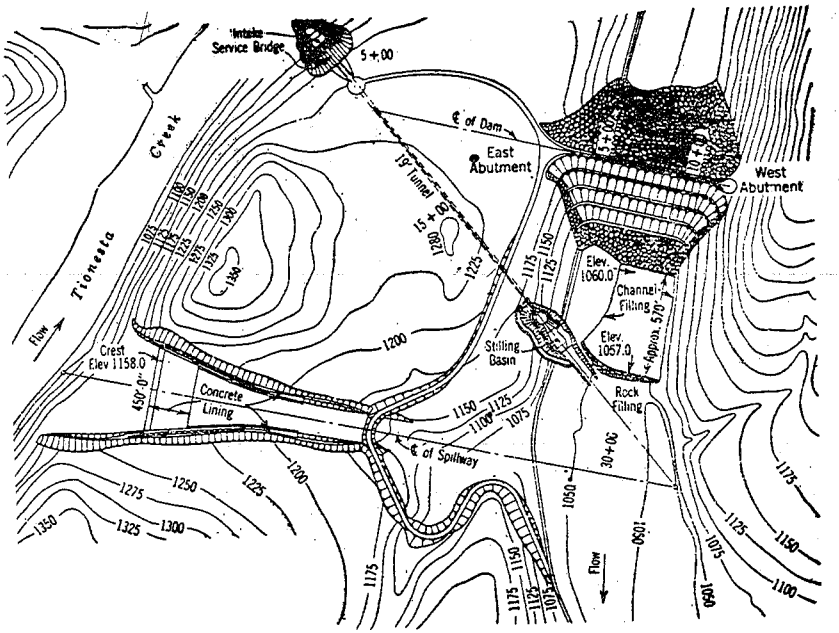


Fig. 21.18. (*e*) Layout Plan of Tionesta Chute Spillway.

21.7.3. Side Walls. The side walls (called training walls) of the chute should be of such a height that water does not spill over them. A sufficient freeboard must, therefore, be provided above the top water nappe, for obtaining the top levels of the side walls. The freeboard is generally given by equation,

$$\text{Freeboard} = 0.61 + 0.04\, V_m \cdot d_m^{1/3} \text{ (M.K.S. or S.I. units)} \quad \ldots(21.9)$$

where V_m = mean velocity of water in the chute reach

d_m = mean depth of water in the chute reach under consideration.

The side walls of the chute may be kept vertical or sloping. But in the vicinity of gated ogee weirs, they will have to be vertical. Generally, a rectangular chute channel is designed.

21.7.4. Design of the Small Ogee Weir Required as a Control Structure for the Chute Spillway. The profile for low ogee weir as recommended by W.E.S. confirms to the following equations for different ratios of H_a/H_e, where H_a is the head due to velocity of approach and H_e is the design head over the crest including velocity head, *i.e.* $(H_e = H_d + H_a)$.

Table 21.7. Table showing equations for d/s profile of a low ogee weir

Values of $\frac{H_a}{H_e}$	$\frac{h}{H_e}$ *range*	*Equation for the d/s profile*
0.0	≥ 1.0	$x^{1.78} = 1.852\, H_e^{0.78} \cdot y$
0.08	1.0 – 0.58	$x^{1.75} = 1.869\, H_e^{0.75} \cdot y$
0.12	0.58 – 0.30	$x^{1.747} = 1.905\, H_e^{0.747} \cdot y$

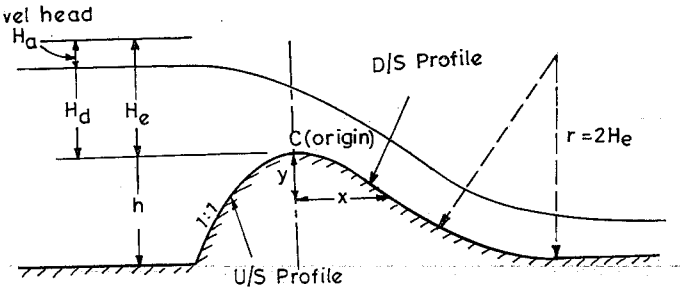


Fig. 21.19. Profile of the small ogee weir to be used as control structure for a chute spillway.

The co-ordinates of the u/s profile, which should merge in a slope of 45° (*i.e.* 1 : 1), as shown in Fig. 21.19, are given in Table 21.8.

Table 21.8. Co-ordinates of u/s profile for low ogee weir

$\frac{x}{H_e}$	$\frac{y}{H_e}$ *for different values of* $\frac{H_a}{H_e}$		
	$\frac{H_a}{H_e}=0.00$	$\frac{H_a}{H_e}=0.08$	$\frac{H_a}{H_e}=0.12$
– 0.000	0.0000	0.0000	0.0000
– 0.020	0.0004	0.0004	0.0004
– 0.060	0.0036	0.0035	0.0035
– 0.100	0.0103	0.0101	0.0099
– 0.120	0.0150	0.0150	0.0147
– 0.140	0.0207	0.0208	0.0199
– 0.150	0.0239	0.0235	0.0231
– 0.160	0.0275	0.0270	0.0265
– 0.175	0.0333	0.0328	0.0325
– 0.190	0.0399	0.0395	0.0390
– 0.195	0.0424	0.0420	—
– 0.200	0.0450	—	—

The reverse curve at the toe may have a radius equal to $2H_e$.

21.7.5. Design of Vertical Curves of the Chute. This is discussed below :

Concave Curve. Whenever the slope of chute changes from steeper to milder [Fig. 21.20 (*a*)], a concave vertical curve shall be provided. In no case, the radius of this curve should be less than 10 *d*, where *d* is the depth of water in metres.

Convex curve. Whenever the slope of the chute changes from milder to sleeper [Fig. 21.20 (*b*)], convex vertical curve shall have to be provided. The curvature should approximate to a parabolic shape given by the equation :

$$q = -x \tan\theta - \frac{x^2}{K\,[4\,(d+h_v)\cos^2\theta]} \qquad ...(21.10)$$

where θ is the slope angle of the floor upstream ;

$(d + h_v)$ is the specific energy of flow at the junction point ;

K is constant which is ≥ 1.5.

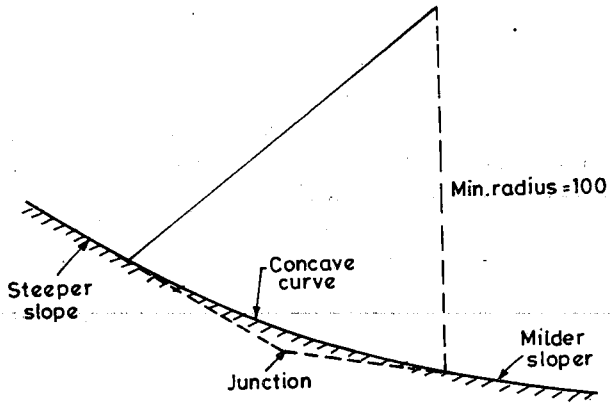


Fig. 21.20 (*a*). Concave Vertical Curve.

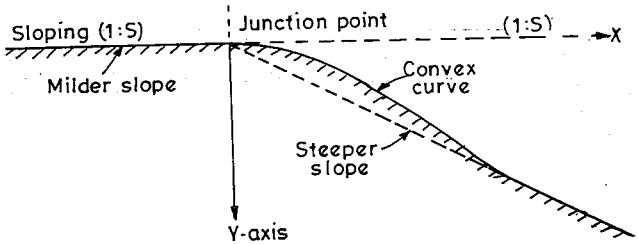


Fig. 21.20 (*b*). Convex Vertical Curve.

21.7.6. Approach Channel of Chute Spillway. An entrance channel called an approach channel, trapezoidal in shape with side slopes 1 : 1, may be constructed so as to lead the reservoir water up to the control structure (low ogee weir). If any curvature (in plan) is required, it is generally confined to the entrance channel, because the velocity of flow is low in this channel. The chute channel called the discharge channel or *discharge carrier* is generally kept straight in plan. If, however, any curvature is required to be provided, its floor should be super elevated to guide the high velocity flow around the bend, thus avoiding piling up of flow towards the outside of the chute. The friction head lost in the entrance channel upto the spillway crest can be calculated by Manning's formula, given as :

$$h_f = S_f \times L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \qquad ...(21.11)$$

where n = Manning's coefficient of roughness
V = Velocity in channel
R = Hydraulic mean depth
L = Length of channel
S_f = Mean energy slope between two points.

The entire chute spillway can hence be divided into the following parts :

(*i*) Entrance channel
(*ii*) Control structure (Low Ogee weir)
(*iii*) Chute channel or Discharge carrier
(*iv*) Energy dissipation arrangements at the bottom in the form of a stilling basin.

The design principles for the various components have already been explained except those for energy dissipation. The energy dissipation arrangements required at the bottom of the spillway shall be explained a little later.

Example 21.2. *Design a suitable profile for a chute spillway with the following data :*

Spillway crest level = 200.0 m

Level of bottom of flank at which the low Ogee weir is to be constructed = 192.0 m

Design discharge = 5,000 cumecs.

Downstream tail water level corresponding to 5,000 cumecs = 103.0 m.

The spillway length consists of 5 spans of 10.0 m clear width each. The thickness of each spillway pier may be assumed to be 3 m. Assume any other data required.

Solution.

Design of approach channel :

$$Q = C \cdot L_e H_e^{3/2}.$$

Assume the coefficient of discharge as 2.18, and taking L_e as the clear width (approximately) $= 10 \times 5 = 50$ m :

and assuming $H_e \approx H$

we get $5{,}000 = 2.18\ (50)\ H^{3/2}$

or $$H^{3/2} = \left(\frac{100}{2.18}\right) = 45.8 \text{ m.}$$

or $$H = (45.8)^{0.667} = 12.8 \text{ m.}$$

Upstream water level = Crest level + H = 200 + 12.8 = 212.8 m.

Bed level of river in flank = 192.0

$\therefore$ Water depth = 212.8 − 192.0 = **20.8 m.**

Assuming the trapezoidal approach channel with 1 : 1 side slopes,

The width of the channel = B

= Total length of spillway = $50 + 4 \times 3 = 62$ m.

Area of channel $= A = (B + y)\ y = (62 + 20.8)\ 20.8 = 1{,}720$ sq. m.

Velocity of approach $= V_a = \dfrac{5{,}000}{1{,}720} = 2.9$ m/sec

Velocity Head $= H_a = \dfrac{V_a^2}{2g} = \dfrac{(2.9)^2}{2 \times 9.81} =$ **0.43 m.**

The wetted perimeter P of the channel

$= [B + 2\sqrt{2}$ (water depth)]

$= [62 + 2\sqrt{2} \times 20.8] = (62 + 58.8) = 120.8$ m.

$$R = \frac{A}{P} = \frac{1{,}720}{120.8} = \textbf{14.2 m.}$$

Assuming the length of the channel to be 160 m ; the head loss due to friction upto the spillway crest, is given by Manning's formula, as :

$$h_f = \frac{n^2 V^2 L}{R^{4/3}} \quad \text{(assume } n \text{ = Manning's rugosity Coefficient = 0.019)}$$

$\therefore$ $$h_f = \frac{(0.019)^2 \times (2.9)^2 \times 160}{(14.2)^{1.33}} = \textbf{0.016 m.}$$

Level of u/s *TEL* = u/s water level + Velocity Head – Head lost up to spillway crest

$= 212.8 + 0.43 - 0.016 = 213.214$; say **213.21 m.**

Hence, head over the crest including velocity of approach

$= H_e = 213.21 - 200.0 = \mathbf{13.21\ m.}$

and $H_d = 13.21 - 0.43 = \mathbf{12.78\ m.}$

Correct Coefficient of Discharge

(*i*) *Correction due to height of weir,*

$$\frac{h}{H_d} = \frac{8}{12.78} = 0.63$$

$$\frac{H_e}{H_d} = \frac{13.21}{12.78} = 1.03$$

From Fig. 21.12 ; $\frac{C}{C_d} = 0.98.$

(*ii*) *Correction due to u/s slope of 45° (i.e. 1 : 1 slope)*

From Fig. 21.13 ; Correction factor = 1.008.

(*iii*) Assuming that the downstream apron elevation is maintained such that it does not affect C_d, we have,

The correct value of coefficient of discharge

$= C = 2.2 \times 0.98 \times 1.008 = 2.17$

Effective length $= L_e = 50 - 2\,[N \cdot K_p + K_a]\,H_e$

assuming $K_p = 0.01$ and $K_a = 0.1$

$$L_e = 50 - 2\,[4 \times 0.01 + 0.1] \times 13.21 = 50 - 3.7 = \mathbf{46.3\ m.}$$

The correct head H_e will then be given by

$$5{,}000 = 2.17 \times 46.4 \times H_e^{3/2}$$

or $$H_e^{3/2} = \frac{5{,}000}{2.17 \times 46.3} = 49.7$$

or $$H_e = (49.7)^{0.667} = \mathbf{13.6\ m.}$$

Hence, corrected $H_d = 13.6 - 0.43 = 13.17$ m ; say 13.2 m.

Hence, the design shall be done for a design head equal to 13.2 m.

Design of Crest Profile

$$\frac{H_a}{H_e} = \frac{0.43}{13.2 + 0.43} = \frac{0.43}{13.63} = 0.0315$$

$$\frac{h}{H_e} = \frac{8}{13.6} = 0.58$$

Since $\frac{H_a}{H_e}$ lies between 0 and 0.08 and $\frac{h}{H_e} \approx 0.58$, we can use the equation $x^{1.75} = 1.869\,H_e^{0.75} \cdot y$ (from Table 21.7).

Hence, the d/s profile is given by the equation

$$x^{1.75} = 1.869\,(13.63)^{0.75} \cdot y$$

or $$x^{1.75} = 13.2\,y$$

or $$y = \frac{x^{1.75}}{13.2} \qquad \ldots(21.12)$$

Position of the d/s apron of spillway

The apron or toe of spillway should be at such an elevation that it does not affect the coefficient of discharge.

$$\text{or} \qquad \frac{h_a + d}{H_e} \geq 1.7$$

$$\text{or} \qquad h_d + d \geq 1.7\,(13.63) = 23.2 \text{ m.}$$

Hence, maximum apron elevation

$$= \text{T.E.L.} - (h_d + d)$$

where T.E.L. $= 212.63 \times 0.016$

$= 213.614$ m ; say **213.61 m**

$\therefore$ Max. apron elevation $= 213.61 - 23.2 = 190.41$ m.

Hence, provide the toe of the spillway (*i.e.* apron level) at *RL* **190.4 m.**

Discharge intensity downstream of spillway piers

$$= q = \frac{5{,}000}{50 + 12} = \frac{5{,}000}{62} = 80.7 \text{ m}^2/\text{sec.}$$

If d is the width, then the velocity $= \dfrac{80.7}{d}$ m/s

$$\text{Specific energy} = d + \frac{V^2}{2g} = g + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81}$$

But specific energy $= 213.614 - 190.4 = 23.214$

$$\therefore \qquad d + \frac{\left(\dfrac{80.7}{d}\right)^2}{2 \times 9.81} = 23.214$$

$$\text{or} \qquad d + \frac{(80.7)^2}{19.62\, d^2} = 23.214$$

$$\text{or} \qquad d^3 + 331 = 23.21\, d^2$$

$$d^3 - 23.21\, d^2 + 331 = 0$$

Solving by hit and trial, we get

$$\boxed{d = 4.2 \text{ m}}$$

The d/s profile is designed between *RL* 200.0 m (crest level) and *RL* = 190.4 (apron level)

Hence, maximum ordinate $y = 200.0 - 190.4 = \mathbf{9.6\ m.}$

Corresponding value of x is obtained from eq. (21.12), as :

$$9.6 = \frac{x^{1.75}}{13.2}$$

$$\text{or} \qquad x^{1.75} = 126.7$$

$$\text{or} \qquad x = (126.7)^{0.57} = 15.08$$

The remaining co-ordinates of d/s profile between $x = 0$ and $x = 15.08$ m are worked out, as given in Table 21.9.

Table 21.9. Coordinates of d/s Profile

x in metres	$y = \frac{x^{1.75}}{13.2}$ *in metres*
0	0
0.5	0.023
1.0	0.077
2.0	0.25
4.0	0.86
6.0	1.74
8.0	2.88
10.0	4.03
12.0	5.84
14.0	7.9
15.08	9.6

The co-ordinates of the u/s profile are calculated in Table 21.10.

Table 21.10. Co-ordinates of u/s Profile
H_e = 13.61 m

$\frac{x}{H_e}$	$\frac{y}{H_e}$	x	y
– 0.000	0.0000	0	0
– 0.020	0.0004	– 0.272	0.0054
– 0.060	0.0035	– 0.817	0.0476
– 0.100	0.0101	– 1.361	0.1375
– 0.120	0.0150	– 1.633	0.2042
– 0.140	0.0208	– 1.905	0.2831
– 0.150	0.0235	– 2.042	0.3198
– 0.160	0.0270	– 2.178	0.3675
– 0.175	0.0328	– 2.382	0.4464
– 0.190	0.0395	– 2.586	0.5375
– 0.200	0.0420	– 2.722	0.5716

The u/s crest is joined at an angle of 45° to the bottom, as shown in Fig. 21.21.

Design of the Chute or Discharge Carrier

The critical depth $y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{(80.7)^2}{9.81}} = 8.77$ m.

The depth at the top of spillway (d) was calculated to be 4.2 m, which is less than y_c. Hence, the flow at the top of the spillway is supercritical.

The chute channel or the discharge carrier should now be given a milder slope for a little distance from toe, but in no case less than the critical slope, so that the flow remains super-critical.

Critical velocity $= V_c = \frac{q}{y_c} = \frac{80.7}{8.77} = 9.18$ m/sec.

But $$V = \frac{1}{n} \cdot R^{2/3} \cdot S^{1/2}$$

A rectangular channel with bottom width 62 m should be provided as the chute channel (discharge carrier) for the chute spillway.

At critical flow, $A = 62 \times 8.77$

$$P = 62 + 8.77 \times 2$$

$\therefore$ $$R = \frac{A}{P} = \frac{62 \times 8.77}{(62 + 8.77 \times 2)} = 6.82$$

Critical slope S_c is, therefore, given as :

$$9.18 = \frac{1}{0.019} (6.82)^{\frac{2}{3}} \cdot S_c^{1/2}$$

or $$S_c^{1/2} = \frac{9.18 \times 0.019}{(6.82)^{2/3}} = \frac{9.18 \times 0.019}{3.61} = \frac{1}{20.7}$$

or $$S_c = \frac{1}{428}$$

A slope of say 1/250 is provided in 25 m distance from the top of the spillway.

Bed level at the end of $\frac{1}{250}$ slope $= 190.4 - \frac{25}{250} = 190.3$ m.

The reverse curve at toe may have a radius equal to

$$2H_e = 2 \times 13.63 = 27.22 \text{ m ; say } 27.5 \text{ m.}$$

From this point onwards, *i.e.* $RL = 190.3$ m and upto RL 103.0 m, the slope of the discharge carrier may be steepened depending upon the site contours. A slope such as 8 : 1 to 4 : 1 may be given in the initial stage and then a steeper slope of say 2 : 1 or 3 : 1 may be given. The entire design is now like that of a rectangular channel. Wherever, the slope is changed, transition curve (convex or concave) may be designed as explained earlier.

In the given question, since the site contours are not known, we shall assume that a slope of 6 : 1 is given for the first 21 m fall (*i.e.* up a RL 169.3) and then a slope of say 2 : 1 is given for the rest of the reach. The depth, velocity, etc. at the end point of 1 in 250 slope may be taken to be the same, as they were at the toe of the spillway, because the small length of 25 m shall not produce much difference. With this assumption, the TEL at the starting point of new slope 6 : 1 ($RL = 190.3$) is equal to $213.21 - 0.1 = 213.11$ m.

The calculations of water depth, velocity, etc. can now be carried out for the entire reach ($RL = 190.3$ to $RL = 103.0$ m) of the discharge carrier by dividing the channel length into small reaches ; say of length 42 m each, as shown in Table 21.11. The depth (d) at the point is assumed in Column (6). Specific energy is calculated in Col. (9). The TEL at the end is calculated in Col. (10). The drop in energy line (h_f) in the interval is calculated in Col. (17). The drop (h_f) at the end of the interval is subtracted from the final calculated TEL at the beginning of interval in Col. (18). The Col. (18), is then compared with Col. (10). They should be almost equal. If difference is large, the assumed depth is changed till equivalence is obtained. The table is otherwise self explanatory.

Table 21.11. Calculations of water depth on chute channel (*i.e.* Discharge Carrier)

S.N.	Distance from start of 6 : 1 slope	Length L	Drop in bed	Bed level	Depth (d) assumed)	Velocity $v = q/d = 80.7/d$	Vel. head $= v^2/2g$	Sp. energy $= d + v^2/2g$	Bed level + Sp. energy = TEL calculated at the end	$A = Bd = 62d$ (Area)	$P = B + 2d = 62 + 2d$ wetted perimeter	$R = A/P$	$R^{4/3}$	$Sf = (n^2 . v^2)/R^{4/3}$; $n = 0.019$	Average Sf	$L . Sf = h_f$	Actual TEL	Froude number $F = V/\sqrt{gd}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	0	0		190.3	4.2	19.2	18.9	23.1	213.4	260	70.4	3.7	5.72	0.0232			213.53	2.97
2	42	42	7	183.3	3.7 3.7 3.62	21.4 22.4 22.3	24.2 25.7 25.3	27.9 29.3 28.92	211.2 212.6 212.22	229 223 224	69.4 69.2 69.24	3.31 3.23 3.24	4.94 4.78 4.8	0.0378 0.0378 0.0374	0.029 0.0305 0.0303	1.22 1.28 1.27	212.18 212.12 212.13	3.74
3	84	42	7	176.3	3.30 3.28	24.45 24.60	30.4 30.8	33.7 34.08	210.0 210.38	205 205	68.6 68.56	2.98 2.98	4.28 4.28	0.0504 0.0505	0.0439 0.0439	1.84 1.84	210.38 210.38	4.33
4	126	42	7	169.3	3.05	26.4	35.7	38.75	208.05	188	68.08	2.77	3.88	0.0652	0.0578	2.43	207.95	4.95
Start of Slope 2 : 1																		
5	168	45	21	148.3	2.5	32.3	53.2	55.7	204.0	155	67	2.32	3.07	0.1224	0.0938	3.94	204.11	6.46
6	210	42	21	127.3	2.21	36.5	67.9	70.11	197.41	136	66.4	2.05	2.56	0.1895	0.1559	6.54	197.46	7.84
7	258.6	48.6	24.3	103.0	1.98 2.015	40.7 40.0	84.7 81.85	86.68 83.865	189.98 186.865	122.6 125.0	66 66.04	1.86 1.895	2.29 2.35	0.259 0.245	0.2242 0.2172	10.9 10.54	186.51 186.87	9.00

Design of Curve No. 1. (At the junction of 250 : 1 and 6 : 1 slopes, a convex curve shall be provided).

The convex curve No. 1 can be designed as per Equation (21.10), as :

$$y = -x \tan\theta - \frac{x^2}{1.5\,[4\,(d + h_v)\cos^2\theta]}$$

$(d + h_v)$ is the sp. energy at the junction point

$$= 4.2 + \frac{\left(\frac{80.7}{4.2}\right)^2}{2 \times 9.81} = 23.21 \text{ m.}$$

$\tan\theta$ = slope of the angle of the floor u/s of the junction point $= \frac{1}{250}$

Since $\tan\theta$ is small, $\cos^2\theta$ will be approximately unity.

$$\therefore \qquad y = -\frac{x}{250} - \frac{x}{6 \times 23.21} = \frac{-x}{250} - \frac{x^2}{139.3}$$

Differentiating w.r. to x, we get

$$\frac{dy}{dx} = \frac{1}{250} - \frac{x}{69.7}$$

The curve meets the downstream slope where

$$\frac{dy}{dx} = -\frac{1}{6}$$ (–ve sign shows that as x increases, y decreases).

Equating, we get $\frac{1}{250} + \frac{x}{69.7} = \frac{1}{6}$

or $$x = 69.7\left(\frac{1}{6} - \frac{1}{250}\right) = 69.7\left[\frac{244}{1{,}500}\right]$$

or **x = 11.3 m.**

Other co-ordinates of this curve for values of x between $x = 0$ and $x = 11.3$ m, determined from the equation $y = \frac{x}{250} + \frac{x^2}{139.3}$ (y is negative of course) are given in table 21.12.

Table 21.12

x in metres	$y = \frac{x}{250} + \frac{x^2}{139.3}$ *in metres*
0	0
1	0.011
3	0.077
5	0.20
7	0.371
9	0.619
10.0	0.78
11.3	0.97

Design of curve No. II (convex). *At the junction of 6 : 1 slope and 2 : 1 slope.*

From Table 21.11, the specific energy at this point is found to be

$$= d + h_v = 3.05 + 35.7 = 38.75 \text{ m}$$

$$\therefore \quad y = -\left\{ x \tan\theta + \frac{x^2}{1.5[4(d+h_v)\cos^2\theta]} \right\}$$

$$\tan\theta = \frac{1}{6}\,;\ \cos\theta \approx 1$$

$$\therefore \quad y = -\left[\frac{x}{6} + \frac{x^2}{6 \times 38.75 \times 1} \right]$$

or $$y = -\left[\frac{x}{6} + \frac{x^2}{232.5} \right]$$

$$\therefore \quad \frac{dy}{dx} = -\left[\frac{1}{6} + \frac{x}{116.25} \right]$$

It meets the downstream slope of 2 : 1 ; $\therefore \ \frac{dy}{dx} = -\frac{1}{2}$ (–ve sign shows that as x increases, y decreases.)

$$\therefore \quad \frac{1}{6} + \frac{x}{116.25} = \frac{1}{2}$$

or $$\frac{x}{116.25} = \frac{1}{2} - \frac{1}{6} = \frac{1}{3}$$

or $$x = \frac{116.25}{3} = 38.75 \text{ m}$$

are $$x = \mathbf{38.75\ m.}$$

It means that this convex curve shall become tangential to the slope of 2 : 1 after traversing a distance of 38.75 m. The co- ordinates of this curve can be found (between $x = 0$ and $x = 38.75$ m), using the equation $y = -\left[\frac{x}{6} + \frac{x^2}{232.5} \right]$ as shown in table 21.13.

Table 21.13

x in metres	$y = \frac{x}{6} + \frac{x^2}{232.5}$ *in metres (y is downward)*
0	0
1	0.17
5	0.95
10	2.10
15	3.47
20	5.05
25	6.85
30	8.87
35	11.10
38.75	12.93

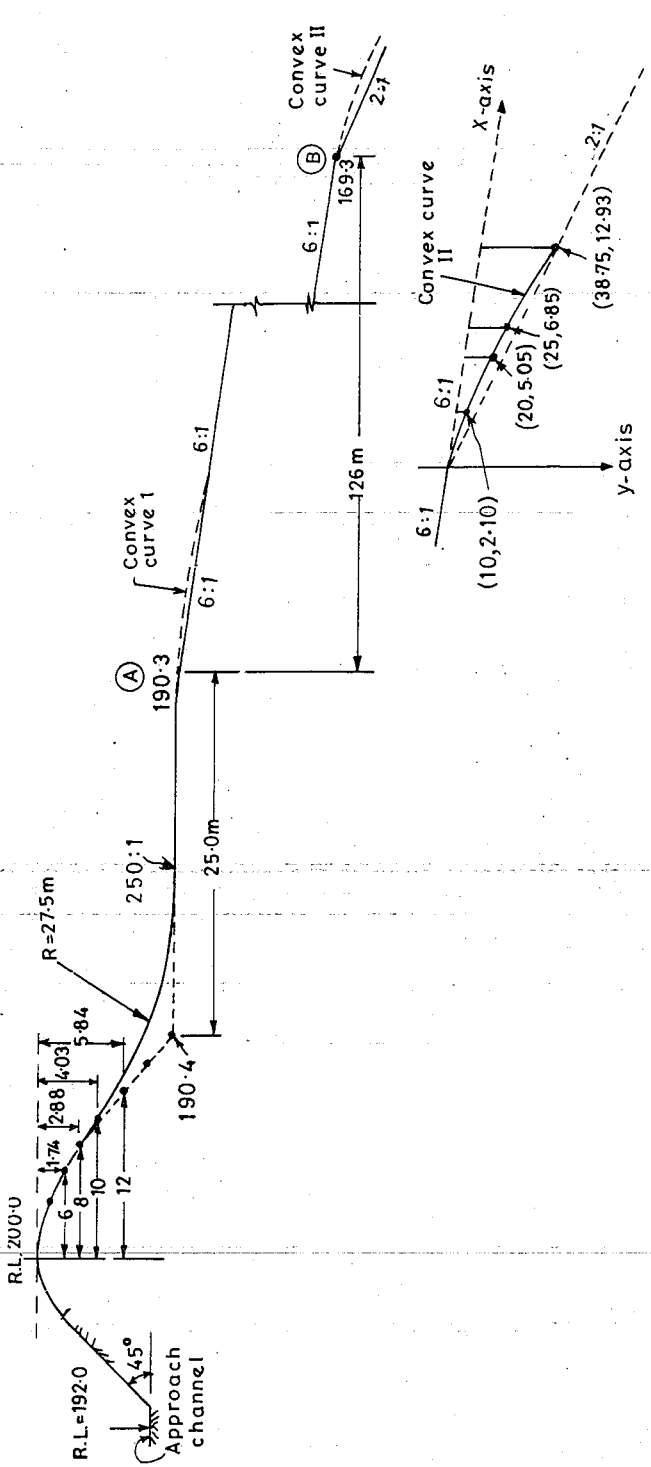


Fig. 21.21. Section of Chute Spillway.

Position of Stilling Basin for Energy Dissipation by Hydraulic Jump (Explained later)*

From Table 21.11, the depth at tail water level

$$d = 2.015 \text{ m}$$
$$F = 9.00$$
$$V = 40 \text{ m/sec.}$$

$\therefore$ $y_1 = 2.015$ m

$V_1 = 40$ m/sec.

$F_1 = 9.0$.

Hence $$y_2 = \frac{y_1}{2}\left[\sqrt{1 + 8F_1^2} - 1\right] = \frac{2.015}{2}\left[\sqrt{1 + 648} - 1\right]$$
$$= 1.0075\,[\sqrt{649} - 1] = \mathbf{24.7\ m.}$$

Provide the basin tail water depth, 50% more than the conjugate depth, as the Froude number is large, or at a depth

$$= 1.05 \times y_2 = 1.05 \times 24.7$$
$$= 25.9 \text{ m say ; } \mathbf{26\ m.}$$

Hence, floor level of jump basin

$$= 103 - 26 = \mathbf{77.0\ m.}$$

U.S.B.R. stilling basin II can be *provided*. This is described in this chapter in article 21.15.

21.8. Side Channel Spillway

The side channel spillway (Fig. 21.22) differs from the chute spillway in the sense that while in a chute spillway, the water flows at right angles to the weir crest after spilling over it, whereas in a side channel spillway the flow of water after spilling over the crest, is turned by 90° such that it flows parallel to the weir crest (AB), as shown in Fig. 21.22 (a).

This type of spillway is provided in narrow valleys where no side flanks of sufficient width to accommodate a chute spillway are available. If a crest length equal to AB is provided along AC (*i.e.*, along axis of a chute spillway), heavy cutting shall be required. In such topographies, a chute spillway may be replaced by a side channel spillway.

The design of side channel, required for diverting the flow, is beyond the scope of this book. However, it may be mentioned that the analysis of flow in the side channel, is made by the application of the momentum principle in the direction of flow. The water entering the side channel has no momentum in the direction in which it has to move. The slope of the side channel should, therefore, be sufficient to overcome friction losses as well as to provide acceleration in the direction of flow against the mass of incoming water.

After the end of the crest A, the water is taken away as in an ordinary chute channel, till it joins the river downstream.

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* Readers are advised to go through this article, only after going through its theory as given in article 21.11.

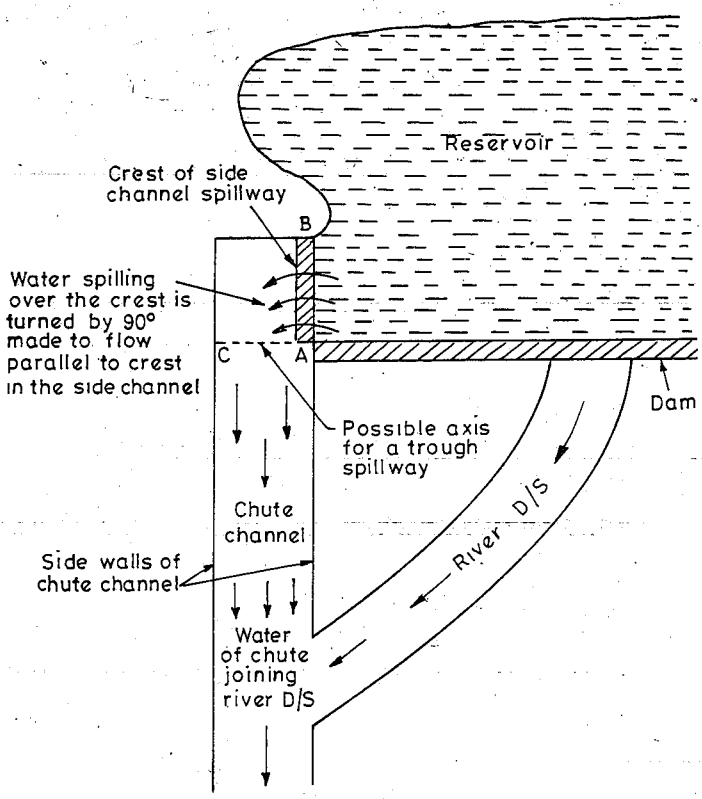


Fig. 21.22. (*a*) Simplified line sketch of a Side Channel Spillway.

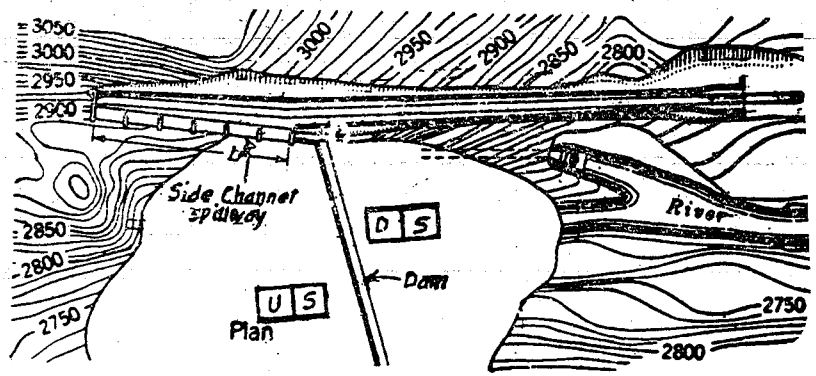


Fig. 21.22. (*b*) Layout plan of a Side Channel Spillway.

Many other spillways may be constructed somewhere in between the chute spillway and the side channel spillway. In such cases, the direction of water after passing over the crest is changed somewhere between 0° and 90°.

21.9. Shaft Spillway

In a shaft spillway (Fig. 21.23), the water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharges the water into the river downstream. Sometimes, the vertical shaft may be excavated through

some natural rocky island or rocky spur existing on the u/s of the river near the dam. Sometimes, artificial shafts may be constructed. For small heights, the shafts may be constructed entirely of metal or concrete, or clay tiles. But for larger heights, reinforced cement concrete may be used. For smaller heights, no special inlet design is necessary, but on large projects, a flared inlet called **morning glory** is often used.

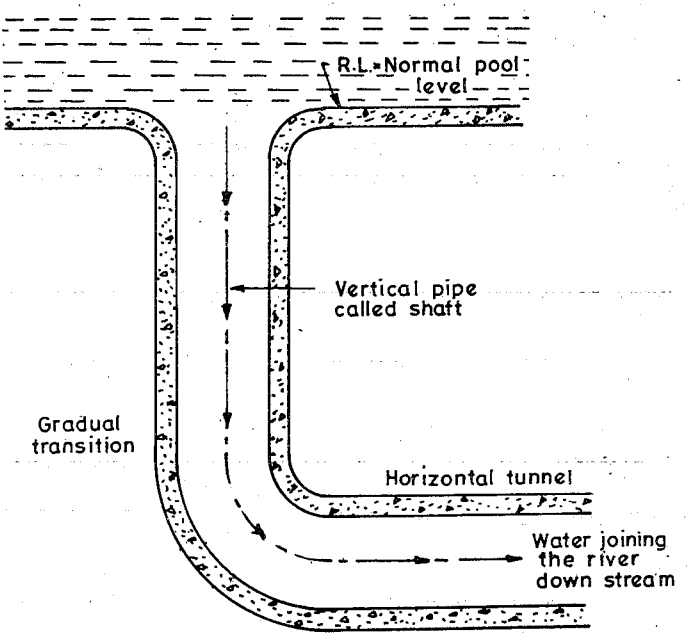


Fig. 21.23. Shaft Spillway.

The horizontal tunnel or the conduit may be taken either through the body of the dam (as may be done in concrete gravity dams) or below the foundations (as may be done in earthen dams). The diversion tunnels constructed for diversion of the river, may sometimes be planned and used for shaft spillways, as shown in Fig. 21.24.

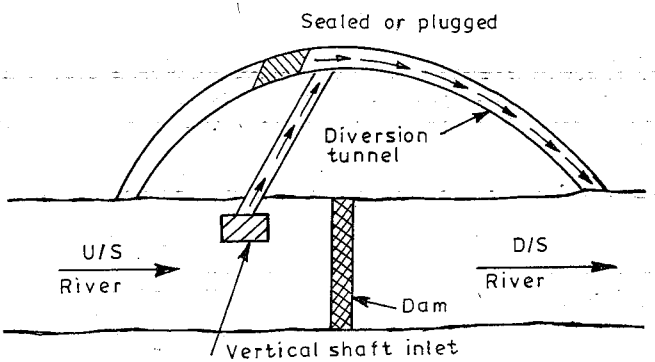


Fig. 21.24. Diversion Tunnel being used as a conduit for the shaft spillway.

A shaft spillway may be adopted when the possibility of an over-flow spillway and a trough spillway has been ruled out because of non-availability of space due to topography. The choice may then lie between a side channel spillway and a shaft spillway. If some suitable rock spur, etc. is available near the reservoir on the upstream, naturally, shaft spillway may become economical and consequently the first choice.

The discharge through the shaft spillway does not increase at such a high rate as it increases in weir type spills. Hence, if the unestimated high floods occur, the shaft spillway may not prove as useful as a weir type spillway (such as overflow, chute or side channel) would have been. Hence, a shaft spillway design should be much more conservative than that of other weir type spillways.

A gradual transition must be provided between the vertical shaft and the horizontal conduit, in order to avoid danger of cavitation. Hydraulic analysis of shaft spillways is difficult and their functioning is generally tested in models. The entry of debris and other materials floating in the reservoir must be prevented from entering the shaft, otherwise, they will clog the shaft or the conduit, especially at the junction point. Properly designed or tested trash racks or floating booms may be provided for this purpose. Since, a shaft spillway is surrounded by water on all sides, it must be connected to the dam or the hill side by a bridge.

21.10. Syphon Spillway

A siphon spillway essentially consists of a siphon pipe, one end of which is kept on the upstream side and is in contact with the reservoir, while the other end discharges water on the downstream side. Two typical installations of siphon pipes are shown in Figs. 21.25 and 21.26.

21.10.1. Tilted Outlet Type of a Syphon Spillway. The siphon pipe in Fig. 21.25 has been installed within the body of the dam. When the valley is very narrow and no space is available for constructing a separate spillway, the siphon pipes can be installed within the dam body, as shown in Fig. 21.25. An air vent may be connected with the siphon pipe. The

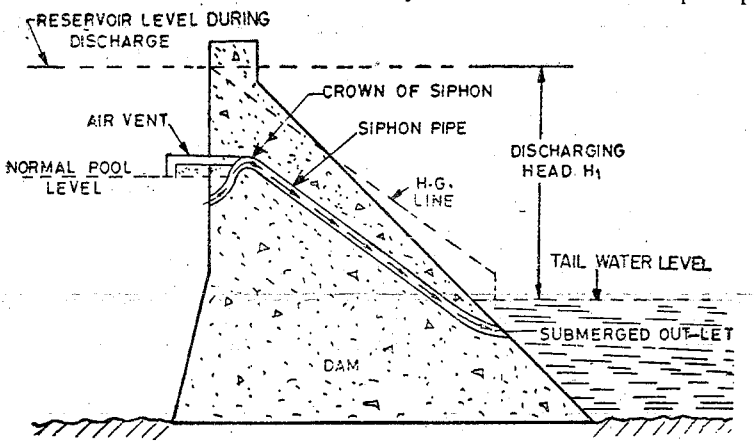


Fig. 21.25. Siphon pipe installed within the gravity dam.

level of the air vent may be kept at normal pool level, while the entry point of the siphon pipe may be kept still lower so as to prevent the entry of debris, etc. in the siphon. The outlet of the siphon may be submerged so as to prevent the entry of the air in the siphon from its d/s end.

When the water in the reservoir is upto or below the normal pool level, air enters the siphon through the vent and siphonic action cannot take place. When once the water level in the reservoir goes above the normal pool level, and if once the siphon is filled with water (*i.e.*, it is primed) ; the water will start flowing through the siphon by siphonic action. The outflow will continue till the water level in the reservoir falls back to normal pool level. As soon as it happens, the air will enter the siphon through the new exposed air vent and the flow will stop.

Discharge formula. The velocity of flow at the outlet of the siphon can be obtained by equating the effective head H_1 (*i.e.*, the difference of water level in the reservoir and the tail water level, for submerged outlet) to the velocity head.

$$\therefore \quad \frac{V^2}{2g} = H_1$$

or $$V = \sqrt{2gH_1}$$

$$\therefore \quad Q = \text{Discharge} = C_d \cdot A \sqrt{2gH_1}.$$

Hence, the discharge through the siphon is given by the equation,

$$Q = C_d A \cdot \sqrt{2gH_1} \qquad ...(21.13)$$

where C_d is the co-efficient of discharge, which is usually about 0.9.

The above equation clearly shows that the discharge, through a siphon spillway is sufficiently independent of the water surface elevation of the reservoir. If the water surface in the reservoir rises, the discharge through the spillway is affected to a less extent because change in H_1 is small as compared to the corresponding change in head over an ogee type spillway. Hence, the discharge through the siphon is nearly always at capacity, when once the water level has risen above the normal pool level. This makes the siphon spillway particularly advantageous in disposing of sudden surges of water, such as may occur in canals and forebays when the outlet gates are closed rapidly. As the rise of water above the crest level is smaller (because of higher discharging capacity even at the start of inflow flood above normal pool level), the height of the non-overflow section of the dam can be kept smaller for the same height of overflow section.

Sometimes, it may not be possible to keep the outlet submerged in tail water because of limitations of negative pressure at the crown. In such a case, the outlet can be left open above the water level, but air entry through the outlet will have to be controlled by priming devices. The effective head H_1, in such a case, will be the difference of reservoir water level and outlet level of siphon.

21.10.2. Hooded Type of a Syphon Spillway. The construction of a Hooded type of siphon spillway [Fig. 21.26 (*a*)] is more commonly adopted. In this case, a reinforced concrete hood is constructed over an ordinary overflow section of a gravity dam. The inlet of this hood is kept submerged so as to prevent the entry of debris, ice, etc. A small depriming hood is kept above the main hood and both these hoods are connected by an

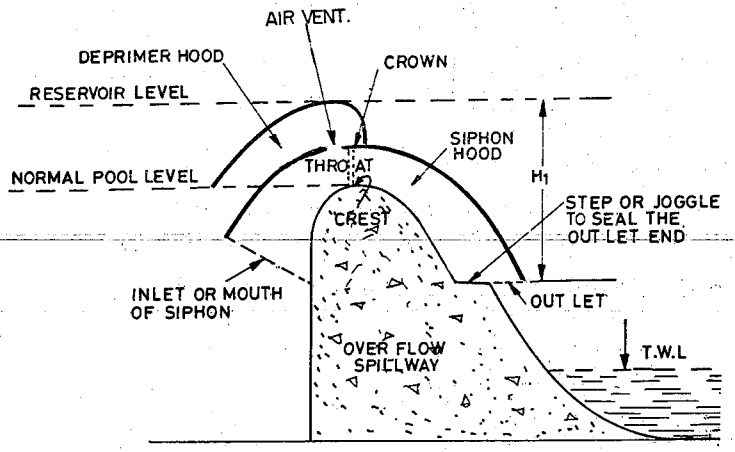


Fig. 21.26. (*a*) Siphon installed over the overflow spillway to increase its effectiveness and discharging capacity.

air vent. The inlet of the deprimer hood is kept at normal pool level. The principle of functioning of both types of installations is essentially the same, except for the initial filling up of the siphon or 'priming'. At normal pool level (*i.e.* full reservoir level), the water stands up to the crest of the spillway. If now a flood enters the reservoir, the water level would start rising and a sheet of water would start flowing over the spillway crest. Since the water level is above the deprimer hood, the air entry at the inlet is sealed. The air entry at the outlet is also sealed by tail water, etc. Hence, the water spilling over the crest, sucks all the remaining air from the hood within minutes. Siphonic action gets established after the air in the bend over the crest have been completely exhausted. A photoview of this type of siphon spillway, called Sarala Sagar spillway, flanked by earth dam constructed on China Vagu stream (a tributary of Krishna river) is shown in Fig. 21.26 (*b*).

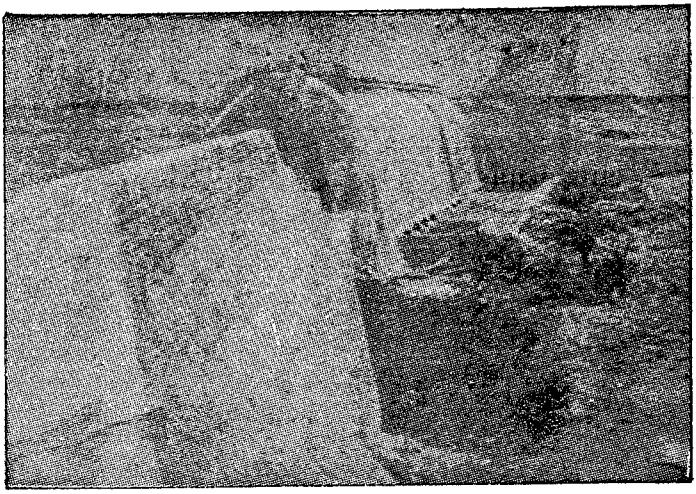


Fig. 21.26 (*b*). Photoview of a Siphon Spillway (Hooded Type).

21.10.3. Priming Devices. When the outlet cannot be sealed by tail water, some other devices, called *priming devices*, are used which lead to an automatic priming of the siphon at a certain rise of water level above the crest. The maximum rise of water level is called *priming depth*. In Fig. 21.26 (*a*), the priming is accomplished by means of a step called *Joggle* which deflects the sheet of water to strike against the lower end of the cover or hood, thus sealing the lower end from the atmosphere. Sometimes, an additional *baby siphon* may be installed as a priming device, as shown in Fig. 21.27.

When the water level reaches slightly above the crest, the baby siphon, which is an additional siphon, starts running full. The sheet of water issuing from it, is arranged to shoot across the lower end of the main siphon, so as to seal it from the atmosphere.

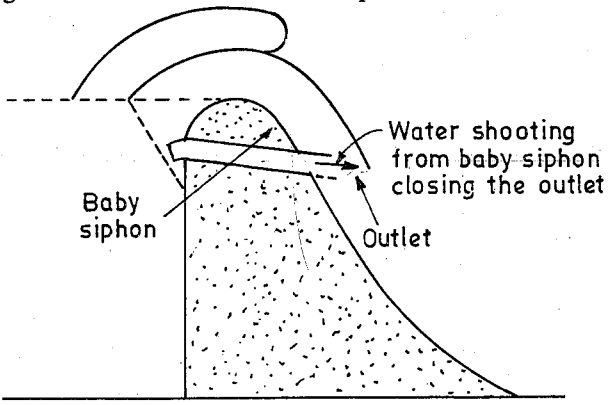


Fig. 21.27. Baby Siphon installed as a priming device.

21.10.4. Negative Pressures. As soon as the siphon is primed, a vacuum is formed at the throat. The negative pressure developed should be limited to such a magnitude that the absolute pressure of water does not exceed the vapour pressure of water at the temperature. This is necessary to avoid cavitation (explained earlier) and its ill effects. Hence, on an average, a maximum negative head equal to 7.5 m or so can be allowed. In other words, the vertical distance from the crown of the siphon (Top point) down to the discharging point (more precisely it is down to the hydraulic gradient line) should not exceed a value of about 7.5 m or so, under average conditions ['∴ atmospheric pressure – vapour pressure = limiting vacuum pressure ; or $10 - 2.5 = 7.5$ m of water'. At high altitudes or in hotter regions, this limit may still go down. This fixes a limit on H_1 and on the discharging capacity of a siphon spillway.

Both types of syphon spillways described above are known as **Saddle siphon spillways**. A special type of siphon spillway called *Volute Siphon Spillway* has been designed in India by Ganesh Iyer and is shown in Fig. 21.28.

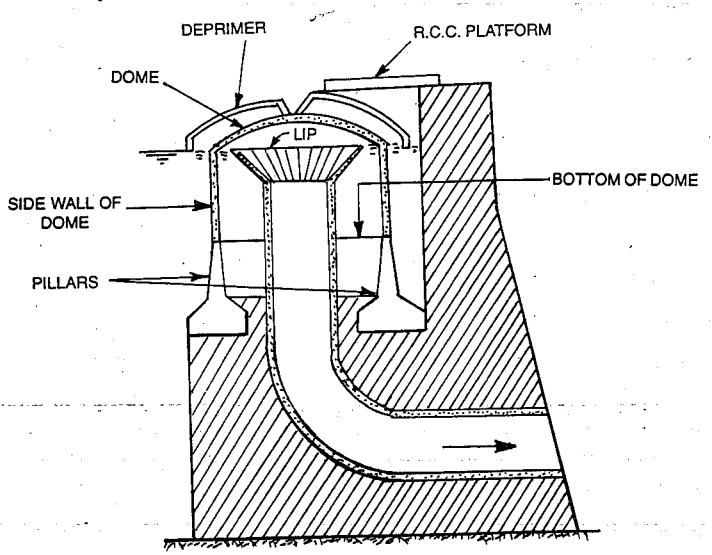


Fig. 21.28. Volute Siphon Spillway.

ENERGY DISSIPATORS

21.11. Energy Dissipation below Overflow Spillways

The water flowing over the spillway acquires a lot of kinetic energy by the time it reaches near the toe of the spillway (because of conversion of potential energy into kinetic energy). If arrangements are not made to dissipate this huge kinetic energy of water, and if the velocity of water is not reduced, large scale scour can take place on the downstream side near the toe of the dam and away from it. These arrangements are known as energy dissipation arrangements or *energy dissipators.*

In general, the kinetic energy of this super-critical flow can be dissipated in two ways :

(*i*) By converting the super critical flow into sub-critical flow by *hydraulic jump.*

(*ii*) By directing the flow of water into air and then making it fall away from the toe of the structure. The energy is dissipated by the *aeration of jet and impact* of water on the river bed. Though some scour will take place, but it is too

small or too far away from the dam to endanger it. Bucket type energy dissipators work on this principle.

21.11.1. Hydraulic Jump Formation. The phenomenon of hydraulic jump has already been explained in details in Chapter 10. It was mentioned therein, that a hydraulic jump can form in a horizontal rectangular channel, when the following relation is satisfied between the pre-jump depth (y_1) and post-jump depth (y_2).

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} \quad \textit{i.e.} \text{ Eq. (10.4)}$$

where q is the discharge intensity.

For a given discharge intensity over a spillway, the depth y_1 is equal to q/V_1 ; and V_1 is determined by the drop H_1, being equal to $\sqrt{2gH_1}$.

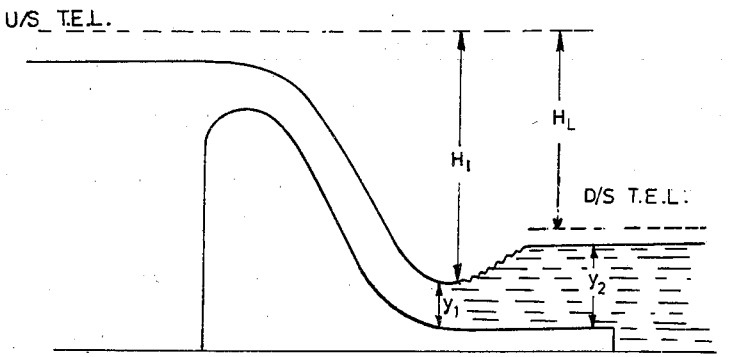
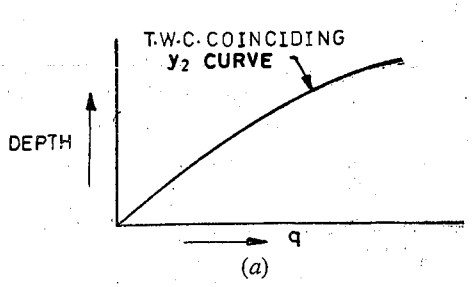


Fig. 21.29

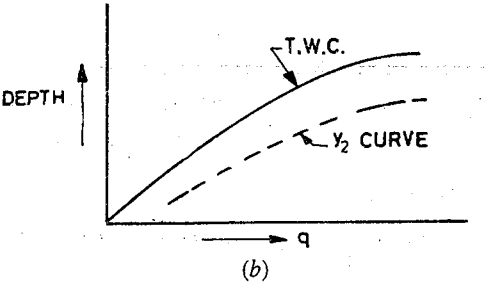
Hence, for a given discharge intensity and given height of spillway, y_1 is fixed and thus y_2 (*i.e.* the depth required for the formation of hydraulic jump) is also fixed. But the availability of a depth equal to y_2 in the channel on the d/s cannot be guaranteed as it depends upon the tail water level, which depends upon the hydraulic dimensions and slope of the river channel below. The problem should, therefore, be analysed before any solution can be found. Hence, for different discharges, the tail water depth is found by actual gauge discharge observations and by hydraulic computations. The post jump depths (y_2) for all those discharges, are also computed from equation (10.4). If a graph is now plotted between q and tail water depth, the curve obtained is known as the *Tail Water Curve (T.W.C.)*. Similarly, if a curve is plotted on the same graph, between q and y_2, the curve obtained is known as the *Jump Height Curve (J.H.C.)* or y_2 *curve.*

Now there are five possibilities

(*a*) T.W.C. coinciding with y_2 curve at all discharges [Fig. 21.30 (*a*)].



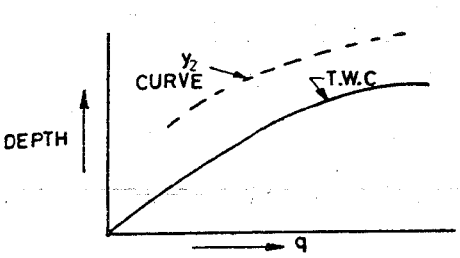
(*a*)



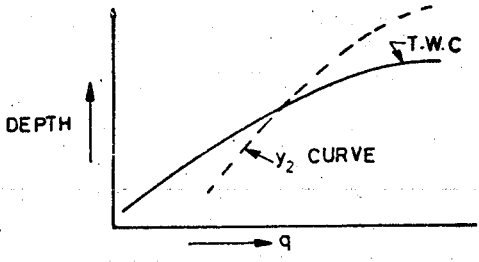
(*b*)

(*b*) T.W.C. lying above the y_2 curve at all discharges [Fig. 21.30 (*b*)].

(*c*) T.W.C. lying below the y_2 curve at all discharges [Fig. 21.30 (*c*)].



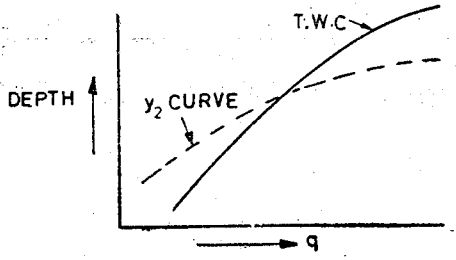
(*c*)



(*d*)

(*d*) T.W.C. lying above the y_2 curve at smaller discharges and lying below the y_2 curve at larger discharges [Fig. 21.30 (*d*)].

(*e*) T.W.C. lying below the y_2 curve at smaller discharges and lying above the y_2 curve at larger discharges [Fig. 21.30 (*e*)].



(*e*)

Fig. 21.30

Depending upon the relative positions of T.W.C. and y_2 curve, the energy dissipation arrangements can be provided below the spillway, as explained below for all these five cases.

21.11.1.1. Energy dissipators for case (*a*) : *When T.W.C. coincides with* y_2 *curve at all discharges.* This is the most ideal condition for jump formation. The hydraulic jump will form at the toe of the spillway at all discharges. In such a case, a simple concrete apron of length $5\,(y_2 - y_1)$ is generally sufficient to provide protection in the region of hydraulic jump, as shown in Fig. 21.31 (*a*).

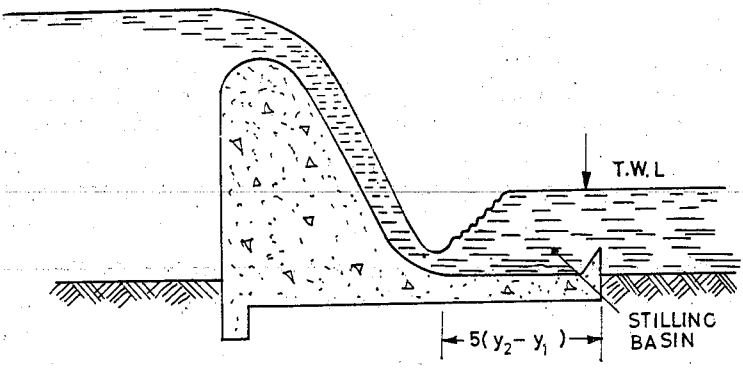


Fig. 21.31. (*a*) Simple horizontal apron.

21.11.1.2. Energy dissipators for case (*b*) : *When T.W.C. is lying above the* y_2 *curve at all discharges.* In this case, when y_2 is always below the tail water, the jump forming at toe will be drowned out by the tail water, and little energy will be dissipated. Water may continue to flow at high velocity along the channel bottom for a considerable distance.

The problem can be solved :

(*i*) by constructing a **sloping apron above the river bed** level as shown in Fig. 21.31 (b_1). The jump will form on the sloping apron where depth equal to y_2 (lesser than the

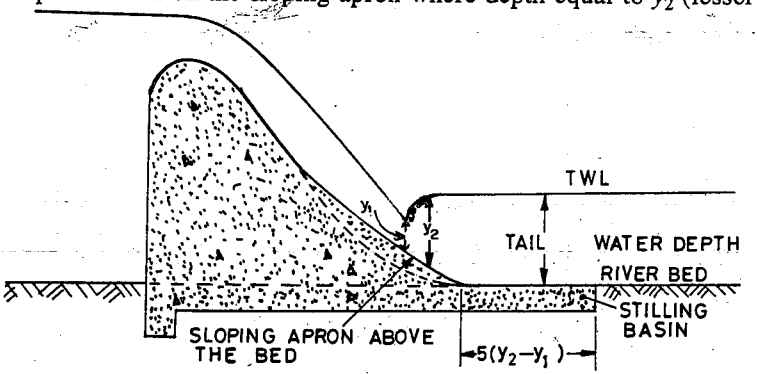


Fig. 21.31. (b_1) Sloping apron above the bed.

tail water depth at toe) is available. The slope of the apron is made in such a way that proper conditions for a jump will occur somewhere on the apron at all discharges. A lot of extra concreting is required to be done, as shown.

(*ii*) A second solution of this problem can be in the form of providing a **roller bucket** type of energy dissipator. It consists of an apron, which is upturned sharply at ends, as shown as in Fig. 21.31 (b_2).

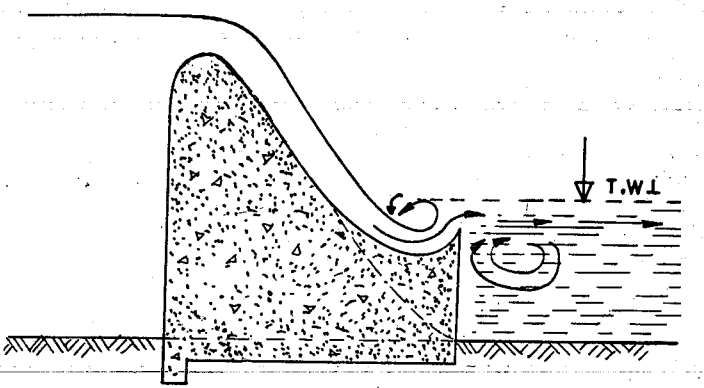


Fig. 21.31. (b_2) Roller Bucket.

Two main rollers are formed which dissipate the energy due to internal turbulence.

The roller which is formed downstream of the bucket, tends to move the scoured bed material towards the dam, thus, preventing serious scour at toe of the dam. Sometimes, the scoured material may enter the bucket under the action of u/s roller, and may cause severe abrasion. A *dentated bucket lip* may, therefore, have to be provided, so as to permit removal of material caught in the bucket.

21.11.1.3. Energy dissipators for case (*c*) : *When T.W.C. lies below the* y_2 *curve at all dischages.* (*i*) If the tail water is very low, the water may shoot up out of the above bucket, and fall harmlessly into the river at some, distance downstream of the bucket. *This bucket is then known as* **ski jump bucket** and can be used for energy dissipation in case (*c*) : *i.e.* when the tail water depth is insufficient or low at all discharges. The ski jump bucket type of an energy dissipator requires sound and rocky river bed, because a part of the energy dissipation takes place by impact, although some of the energy is dissipated in air by diffusion and aeration.

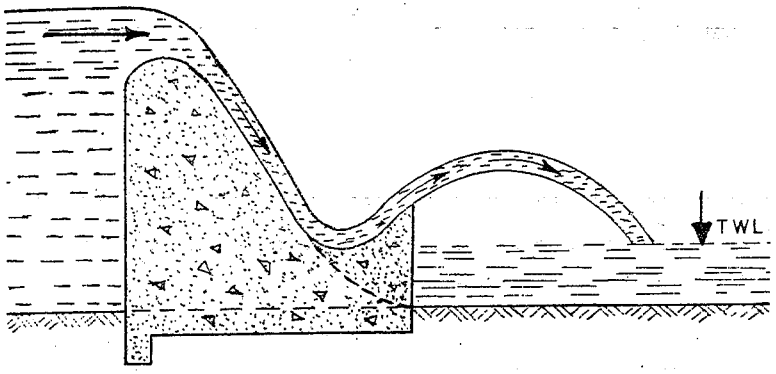


Fig. 21.31. (c_1) Ski jump bucket.

(*ii*) The second solution to the problem can be the *provision of a* **sloping apron** as *in case (b) but* **below the river bed,** as shown in Fig. 21.31 (c_2). The required depth y_2

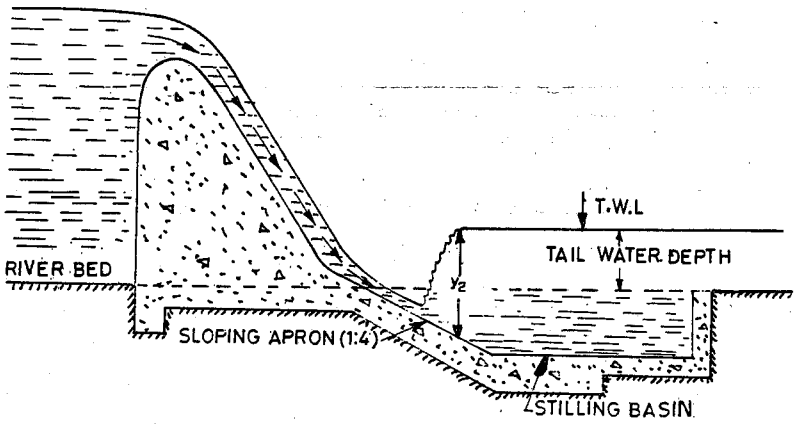


Fig. 21.31. (c_2) Sloping apron below the bed.

which is greater than T.W. depth, can thus be made available by letting the jump form on this sloping apron as shown. This sloping apron and the horizontal cistern of length $5(y_2 - y_1)$ shall be entirely in cutting and may be expensive, though otherwise quite satiactory.

(*iii*) The third solution to this problem may be the construction of a **subsidiary dam** below the main dam, so as to increase the tail water depth and cause a jump to form at the toe of the main dam, as shown in Fig. 21.31 (c_3).

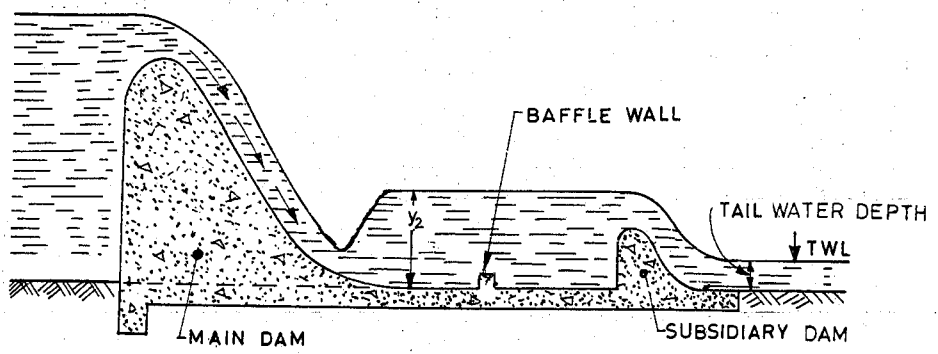


Fig. 21.31. (c_3) Subsidiary dam construction.

If the tail water defficiency is small, a **baffle wall** or a **row of friction blocks** may be provided so as to dissipate the residual energy. The baffles, generally give way, under high velocity jets due to their cavitation effects, and hence, are suitable only for low spillways or weirs. They should be sufficiently strong to withstand impact from ice and floating debris. The location, shape, size and spacing of these baffles can be best determined by model studies. Their use for weirs and canal falls have already been explained in the earlier chapters.

21.11.1.4. Energy dissipators for case (*d*). *When T.W.C. lies above the y_2 curve at low discharges and lies below the y_2 curve at high discharges.* In this case, at low discharges, the jump will be drowned and at high discharges, tail water depth is insufficient. The solution to the problem lies in providing a **sloping apron partly above and partly below the river bed** as shown in Fig. 21.31 (*d*). The horizontal apron and end sill should also be provided.

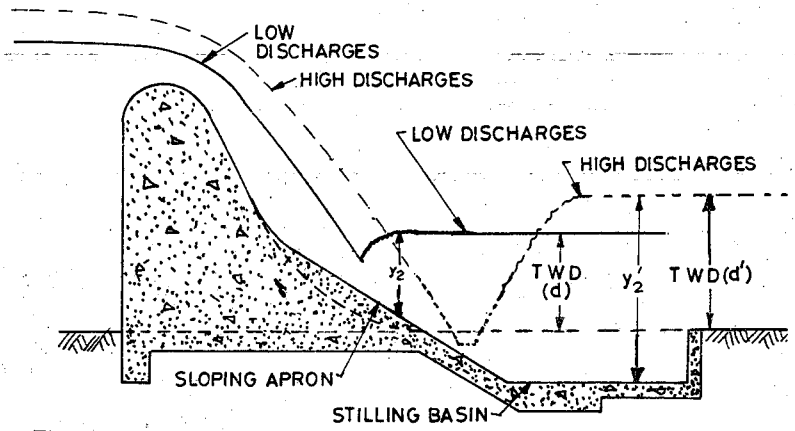


Fig. 21.31. (*d*) Sloping apron partly above and partly below the ground level.

At low discharges, the jump will form on the apron above the river bed, where the available depth is equal to the required depth and less than the T.W. depth. Similarly, at high discharges, the jump will form on the apron below the river bed, where the available depth is more than the T.W. depth and equal to the depth required for jump formation.

21.11.1.5. Energy dissipators for case (*e*). *When tail water depth is insufficient at low discharges and is greater at high discharges.* This case is just the reverse of case (*d*)

and the same arrangement which was made in case (*d*) will serve the purpose. The only difference will be that at low discharges, the jump will form on the apron below the bed; and at high discharges, the jump will form on the apron at a point above the bed.

21.12. Energy Dissipation Below Other Types of Spillways

The five hydraulic conditions between tail water depth and post jump depth (y_2) discussed in the previous article, can occur below any type of spillway or even below the sluiceways or dam outlets, and erosion control can be obtained by the methods described. A chute or a shaft or a side channel spillway generally discharges water at a point far away from the dam. Hence, the protection is required only for the spillway, as the danger to the main dam is not there. Due to this reason, a hydraulic stilling basin is generally sufficient, and may be provided at the discharging point of the spillway. If sound rocks are available, a Ski jump bucket may be provided at low cost.

21.13. Energy Dissipation Below Sluiceways or Dam Outlets

Discharge through the sluiceways is generally small, and less protection is required below them. The water from the sluiceway is often allowed to fall directly into the stilling basin. Sometimes, the jet may be spread by a *jet deflector* at the outlet. Sometimes the energy dissipation arrangement of the main spillway may be utilised for sluiceways also.

21.14. Use of Hydraulic Jump as Energy Dissipator and Design of Stilling Basins

It was stated in chapter 10, that the hydraulic jump formation depends considerably upon the Froude number of the incoming flow (F_1). The pre-jump depth (y_1) and post-jump depth (y_2) are also governed by the equation (10.6) [derived in chapter 10] as:

$$\frac{y_2}{y_1} = \frac{1}{2}\left[\sqrt{1 + 8F_1^2} - 1\right] \qquad ...(10.6)$$

$$\text{where } F_1 = \frac{V_1}{\sqrt{g \cdot y_1}}$$

It was also stated in the same chapter, that the energy dissipation in the jump, depends upon the Froude number. Different types of jump were described for different values of Froude numbers. If this incoming Froude number F_1 is higher, the greater energy dissipation can take place. The approximate percentage loss of energy for various values of F_1 are given in Table 21.14.

Table 21.14

F_1	*% loss of energy*
2.5	17
4.5	45
9.0	70
14.0	80
20.0	85

However, the real problem in the design of stilling basins, is not the absolute dissipation of energy, but is the dissipation of this energy in as short a length as possible.

Hence, the effectiveness of a hydraulic jump has to be viewed in this light. The types of jumps for different ranges of Froude numbers, given in chapter 10, are again summarised below :

(*i*) When $F_1 < 2.5$, the jump is weak and energy loss is low. No blocks or other devices are provided in this range.

(*ii*) When F_1 lies between 2.5 to 4.5, as in case of weirs and barrages, the jump is trouble-some and oscillating, which gives rise to heavy waves on the surface. *Wave Suppressors* may be needed in this range. The length of jump may be taken as $5\,(y_2 - y_1)$.

(*iii*) *When F_1 lies between 4.5 and 9.0 ; as is generally the case for dam spillways,* the jump performance is at its best. The jump is then called a steady jump. The length of the jump is almost constant and equal to $6y_2$ in this range. Hence for large spillways, where y_2 may be quite high, very long and expensive stilling basins may be required. Some auxiliary devices may also be introduced for further stabilising the flow and to reduce the length of the basin.

(*iv*) When $F_1 = 9.0$ and larger ; it is a strong jump. But the jump is likely to be rough and choppy. Hence, a bucket type of energy dissipator is preferred to a stilling basin of hydraulic jump type.

21.15. Standard Stilling Basins

Various types of stilling basins have been generalised for use on different types of works, by various agencies. The designs of these basins have been developed on the basis of long experience and on model studies, keeping in view the protection obtained consistent with economy. These basins are not simple concrete aprons but are generally provided with **auxiliary devices** such as chute blocks, sills, baffle walls, etc. These devices can help in dissipating the energy of flow by offering resistance to flow and may stabilise the flow in a shorter length of the basin, thus affecting economy.

In general, a stilling basin may be defined, as a structure in which the energy dissipating action is confined. If the phenomenon of hydraulic jump is basically used for dissipating this energy, it may be called a hydraulic jump type of stilling basin. The auxiliary devices may be used as additional measures for controlling the jump, etc.

Before we reproduce a few standard stilling basins, let us first describe, in brief, the effects produced by auxiliary devices.

Chute Blocks. Chute blocks are a kind of serrated device (*i.e.* row of small projections like teeth of saw) and provided at the entrance of the stilling basin. The incoming jet of water is furrowed and partly lifted from the floor, producing a shorter length of jump than what would have been without them. They also help in stabilising the flow and thus improve the jump performance (Fig. 21.32).

Sills and Dentated Sills. Sill or more preferably dentated sill is generally provided at the end of the stilling basin. The dentated sill diffuses the residual portion of high velocity jet reaching the end of the basin. They, therefore, help in dissipating residual energy and to reduce the length of the jump or the basin (Refer Fig. 21.32).

Baffle Piers. They are the blocks placed within the basin, across the basin floor. They help in breaking the flow and dissipate energy mostly by impact. These baffle piers, sometimes called **friction blocks,** are very useful in small structures, such as low

spillways and weirs, etc. They, however, give way due to cavitation, under the influence of high velocity jets, and hence are unsuitable for large works.

21.15.1. U.S.B.R. Basins. U.S.B.R. has standardised stilling basins for different ranges of Froude numbers. The important of these basins, are :

(1) **U.S.B.R. stilling basin II.** This is recommended for use on large structures, such as dam spillways, large canal structures, etc., when the incoming *Froude number* (F_1) *is more than 4.5.* The dimensions of the chute blocks, dentated sill, etc. are shown in Fig. 21.32. The length of the basin is related to the Froude number (F_1) as given in Table 21.15.

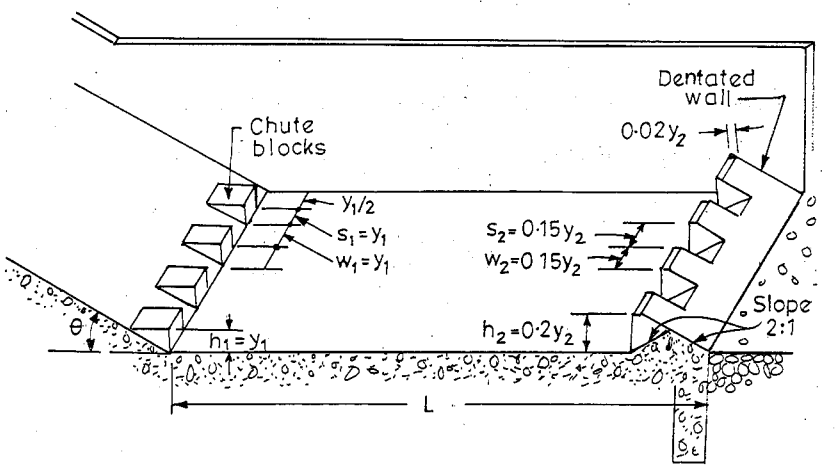


Fig. 21.32. U.S.B.R. Stilling basin II ($F_1 > 4.5$).

Table 21.15

F_1	*Length of the basin*
4	$3.6\ y_2$
6	$4\ y_2$
8	$4.2\ y_2$
10 or more	$4.3\ y_2$

An economy in the length of the basin up to about 35% ($4.3\ y_2$ in place of $6y_2$) is thus obtained with auxiliary devices. The floor of the basin should be set at such a level as to provide 5% more water depth than y_2.

(2) **U.B.S.R. stilling basin IV.** This type of stilling basin is shown in Fig. 21.33. It is used for Froude number varying between 2.5 and 4.5, which generally occurs in canal weirs, canal falls, diversion dams, etc. This basin is applicable only to rectangular cross sections. Since oscillating waves are generated in this range of Froude number, they are tried to be controlled at source by providing large chute blocks. The spacing between the blocks may be further reduced than what is shown in Fig. 21.33 (up to say 0.75 W) for better hydraulic performance. The floor of the basin may be set at such a level as to

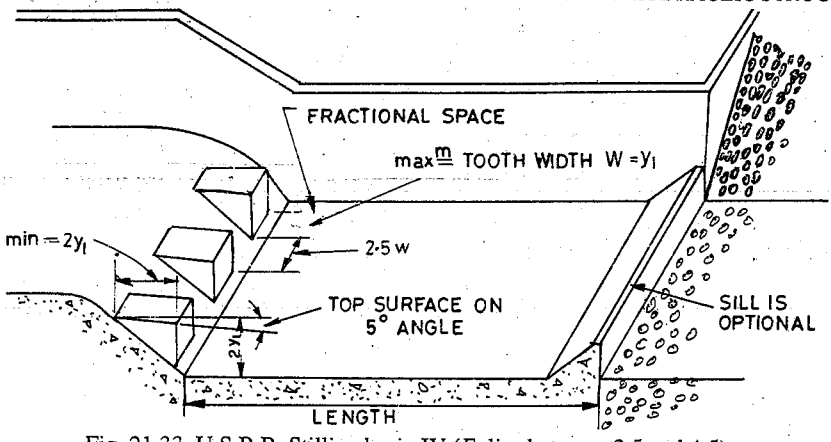


Fig. 21.33. U.S.B.R. Stilling basin IV (F_1 lies between 2.5 and 4.5).

provide 5 to 10% more water depth than the theoretical y_1. The length of the basin is generally kept equal to 5 $(y_2 - y_1)$.

21.15.2. I.S.I. Standardised Basins. Indian Standard Institution has also standardised certain stilling basins for uses under different conditions. A full description of such basins is available in IS : 4997 – 1968. Stilling basin I and II (for $F_1 < 4.5$ and $F_1 > 4.5$ respectively) with horizontal aprons and stilling basins III and IV with sloping aprons, have been standardised and described in the above reference. Students or field staff may refer to this I.S.I. publication in their particular needs.

21.16. Dynamic Force on Spillway Bucket and Spillway Bottom

When water flows over the curved bottom portion of an ogee spillway, there is a continuous change of velocity. Due to this change in velocity, there is a change in momentum (momentum = mass × velocity). Hence, a force is exerted on the spillway in accordance with Newton's second law of motion, *i.e.* rate of change of momentum is equal to this impressed force. This force is known as *dynamic force.* The spillway bottom where this curvature is provided is subjected to this force. The evaluation of this force is, therefore, necessary in the design of the bottom portion, or the design of the bucket, if it is provided as energy dissipator in an overflow spillway.

The magnitude of this force can be easily determined by using Newton's second law and by resolving all the forces acting on the curved element of water *AB* [Fig. 21.34 (*a*)] in horizontal and vertical directions. Mass of water when multiplied by net change in velocity in one direction will give the net component of this dynamic force in that direction. If F is this dynamic force, then

$$F_{(H)} = [V_{2(H)} - V_{1(H)}] \times \text{mass of water in the element}$$

$$F_{(V)} = [V_{2(V)} - V_{1(V)}] \times \text{mass of water in the element}$$

$$F = \sqrt{F_{(H)}^2 + F_{(V)}^2}$$

where V_1 and V_2 are the velocities at the entrance and exit of the curved element. $V_{1(H)}$ and $V_{2(H)}$ are their horizontal components, and $V_{1(V)}$ and $V_{2(V)}$ are their vertical components. An illustrative example has been solved below :

Example 21.3. *An overflow ogee spillway of height 15 m is discharging water with a head of 2 m over the crest. A reverse curvature of radius 4 m, subtending an angle of 60° at the centre, is provided at the spillway bottom as shown in Fig. 21.34 (a). Assuming the discharge coefficient for the spillway as 2.2, determine the magnitude of the dynamic force on the curve portion AB of the spillway.*

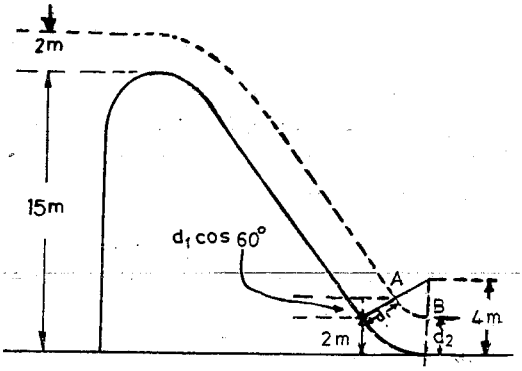


Fig. 21.34 (*a*)

Solution. The discharge over the spillway is given as :

$$Q = C \cdot L \cdot H^{3/2}$$

Discharge per unit length,

$$q = C \cdot H^{3/2}$$

$$= 2.2 \times (2)^{3/2} = 6.23 \text{ cumecs/m}$$

Let d_1 and d_2 be the water depths, V_1 and V_2 be the flow velocities at A and B respectively.

Neglecting any loss of energy and the head due to velocity of approach and applying Bernoulli's theorem between u/s water surface and at sections A and B, respectively we, get

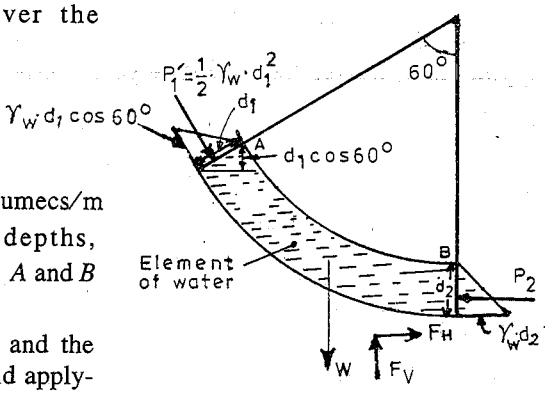


Fig. 21.34 (*b*)

$$15 + 2 = 2 + d_1 \cos 60° + \frac{V_1^2}{2g} = d_2 + \frac{V_2^2}{2g}$$

But $V_1 = \frac{q}{d_1}$; and $V_2 = \frac{q}{d_2}$

$$\therefore \quad 17 = 2 + d_1 \cos 60° + \frac{q^2}{2gd_1^2} = d_2 + \frac{q^2}{2gd_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{(6.23)^2}{2 \times 9.81\, d_1^2} = d_2 + \frac{(6.23)^2}{2 \times 9.81\, d_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} = d_2 + \frac{1.97}{d_2^2}$$

or $$17 = 2 + \frac{d_1}{2} + \frac{1.97}{d_1^2} \qquad ...(i)$$

and $$17 = d_2 + \frac{1.97}{d_2^2} \qquad ...(ii)$$

$\therefore$ (*i*) yields :

$$15 = \frac{d_1}{2} + \frac{1.97}{d_1^2}$$

or $$30d_1^2 = d_1^3 + 3.94$$

Solving by hit and trial, we get

$$\boxed{d_1 = 0.365 \text{ m}}$$

Similarly, solving equation (*ii*), we get

$$17 = d_2 + \frac{1.97}{d_2^2}$$

or $$17d_2^2 = d_2^3 + 1.97$$

by hit and trial, we get $\boxed{d_2 = 0.344 \text{ m}}$

Now the hydrostatic forces P_1 and P_2 per unit length are given as :

$$P_1 = \tfrac{1}{2} \cdot \rho_w \cdot d_1^2 \cos 60° = \tfrac{1}{2} \times 9.81 \times (0.365)^2 \times \tfrac{1}{2} = 0.327 \text{ kN/m run}$$

$$P_2 = \tfrac{1}{2} \cdot \rho_w \cdot d_2^2 = \tfrac{1}{2} \times 9.81 \times (0.344)^2 = 0.58 \text{ kN/m run}$$

Resolving all the Forces in the Horizontal Direction

Algebraic sum of all the forces in the horizontal direction

$$= P_1 \cos 60° + F_H - P_2 \qquad ...(iii)$$

The rate of change of momentum per unit length in the horizontal direction

$$= \frac{\rho_w \cdot q}{g} [V_2 - V_1 \cos 60°]$$

$$= \frac{\rho_w \cdot q}{g} \left[V_2 - \frac{V_1}{2} \right] \qquad ...(iv)$$

But $$V_1 = \frac{q}{d_1} = \frac{6.23}{0.365} = 17.06 \text{ m/sec.} ; \text{ and } V_2 = \frac{q}{d_2} = \frac{6.23}{0.344} = 18.12 \text{ m/sec.}$$

Equating (*iii*) and (*iv*), we get

$$P_1 \cos 60° + F_H - P_2 = \frac{\rho_w q}{g} \left[V_2 - \frac{V_1}{2} \right]$$

or $$0.327 \times \tfrac{1}{2} + F_H - 0.58 = \frac{9.81 \times 6.23}{9.81} \left[18.12 - \frac{17.06}{2} \right] = 59.75$$

or $$F_H = \mathbf{60.16 \text{ kN}}$$

Absence of –ve sign indicates that the assumed direction of F_H is correct.

Resolving all the Forces in the Vertical Direction

The weight W of the water in the curved element AB

$$= \left[\frac{2\pi \cdot R}{360°} \times \theta° \right] \cdot y_{mean} (\rho_w)$$

$$= \left[\frac{2\pi \times 4}{360°} \times 60°\right] + \left[\frac{0.365 + 0.344}{2}\right] \times 9.81 = 14.57 \text{ kN}$$

Resolving all the forces in the vertical direction, we get

$$W + P_1 \sin 60° - F_V = \frac{\gamma_w \cdot q}{g} [0 - V_1 \sin 60°]$$

or $$14.57 + 0.327 \times \frac{\sqrt{3}}{2} - F_V = \frac{9.81 \times 6.23}{9.81} \left[-17.06 \sin 60°\right]$$

$$14.85 - F_V = -92.04$$

or $$F_V = \mathbf{106.89\ kN}$$

Absence of –ve sign indicates that the assumed direction of F_V is correct and is vertically upward.

Resultant force $= \sqrt{F_H^2 + F_V^2} = \sqrt{(60.16)^2 + (106.89)^2}$

$= \mathbf{122.64\ kN.}$

Hence, the resultant dynamic force is equal to 122.64 kN and acts at an angle $= \tan^{-1}\left(\frac{106.89}{60.16}\right) = 60.6°$ with the horizontal in the upward direction, as shown in Fig. 21.34 (*c*).

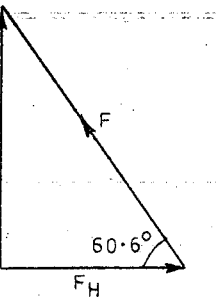


Fig. 21.34 (*c*)

SPILLWAY CREST GATES

If a temporary barrier can be installed over the permanent raised crest of a spillway, additional water can be stored between the spillway crest and the top of the barrier during the fag-end of the rainy season. The small flows in excess of the barrier top level, may be permitted to pass over the barrier. If, however, large flood occurs, the barrier may be removed and full spillway capacity made available for the outflow.

Sometimes on large dams, regular gates may be installed over the permanent crest, so as to function like a movable additional crest. In such a case, the height of the permanent raised crest can be reduced and the balance provided by the movable crest (*i.e.* gate). If there is a permanent raised crest up to the gate top, the storage, of course would be equal to that of a gated crest ; but in times of serious floods, the rise in flood level would be much more as compared to what would have been in a gated crest. This is because, the gates would be opened during serious floods so as to provide more head and hence larger discharge and consequent lesser rise in flood levels. Hence, the top level of the non-overflow section and the value of land acquisition for the reservoir which has to be determined by the maximum rise of flood above the spillway crest, can be reduced by providing gated crest or controlled crests. In other words, the dam height can be reduced for the same useful storage, or more useful storage can be obtained for the same height, provided the dam spillway is controlled by gates, etc.

This saving in the dam height and land acquisition will be more, if more height of the gates is provided. This saving is, however, counter-balanced by the cost of the gates. The cost of the gates would be more if their height is increased. (The gate cost include the principal cost and OMR, *i.e.* operational, maintenance and repair costs.) An economic balance between these two factors must be worked out and the cheapest combination found before deciding the height of the permanent crest and the height of

the temporary crest (*i.e.* gates). This is also governed by the limitations of maximum available gate heights.

Gates can be provided on all types of spillways except siphon spillways. In siphon spillways, the gates are not required as the rise in flood level is already small compared to other types of spillways. The gates for earthen dams should be provided with caution, since the faulty operation or failure of their operation may lead to serious rise in flood levels, causing overtopping and failure of dam.

21.17. Types of Spillway Gates

The various types of spillway gates are described below in brief :

21.17.1. Dropping Shutters or Permanent Flash Boards. They consist of wooden panels usually 1.0 to 1.25 m high. They are hinged at the bottom and are supported against the water pressure by struts (Fig. 21.35). The shutters fall flat on the crest when the downstream supporting struts are tripped. Hence they are not suitable for curved crests.

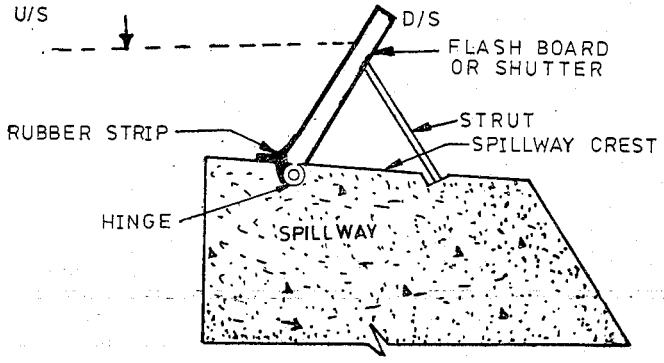


Fig. 21.35. Flash board or Dropping shutter.

These shutters can be raised or lowered from an overhead cableway or a bridge. Various types of shutters which drop and hoist themselves automatically, have been designed these days. These automatic shutters work on the principle of counter weights acting against the water pressure. Automatic shutters do not function well when interferred by floating debris, ice, etc.

Sometimes temporary flash boards, which shall fall as soon as overtopped by water, may be used for very minor works.

All kinds of flash-boards do have some disadvantages and hence used only on small spillways of minor importance.

21.17.2. Stop Logs and Needles. Stop logs consist of wooden beams or planks placed one upon the other and spanning in the grooves between the spillway piers (Fig. 21.36). They can be placed and removed either by hand or with hoisting mechanism. Considerable time may get wasted in removing them, if they become jammed in the slots. Leakage between the logs is also a big problem. They are, therefore, used on very minor works.

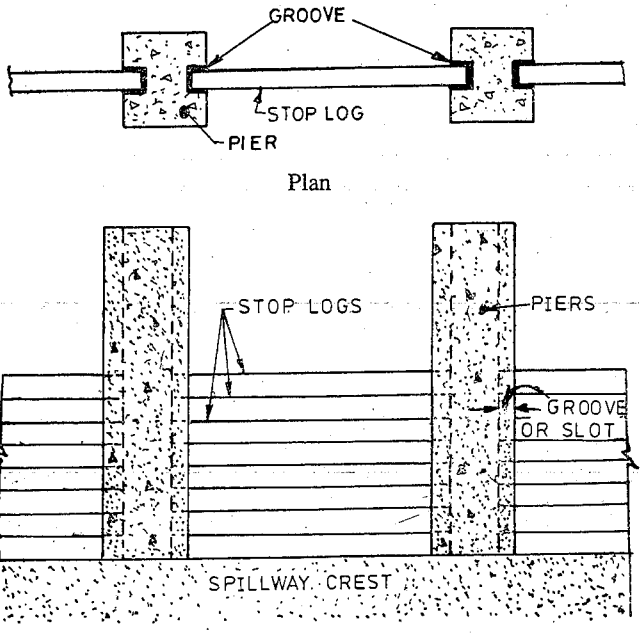


Fig. 21.36. Stop logs.

Needles. Needles are wooden logs kept side by side with their lower ends resting in a keyway on the spillway and upper ends supported by a bridge (Fig. 21.37). It is very difficult to handle these needles at the time of flow and hence they are not used on any major works. They are sometimes used for emergency bulk heads, where they need not be replaced until the flow has stopped.

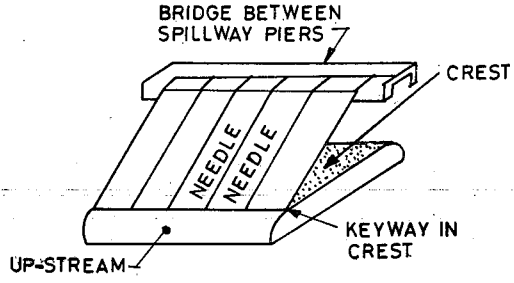
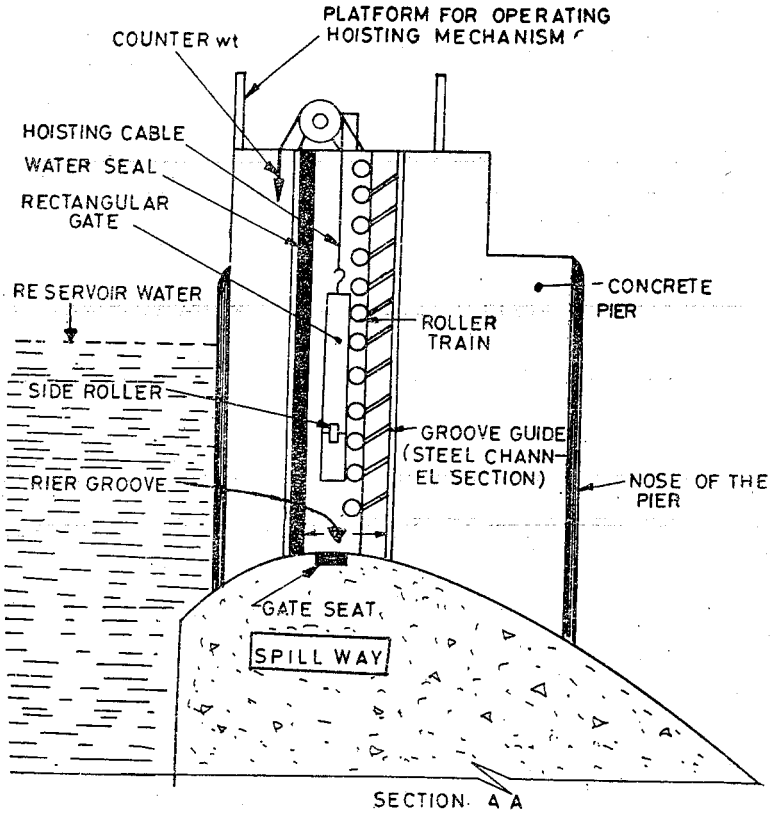


Fig. 21.37. Needles.

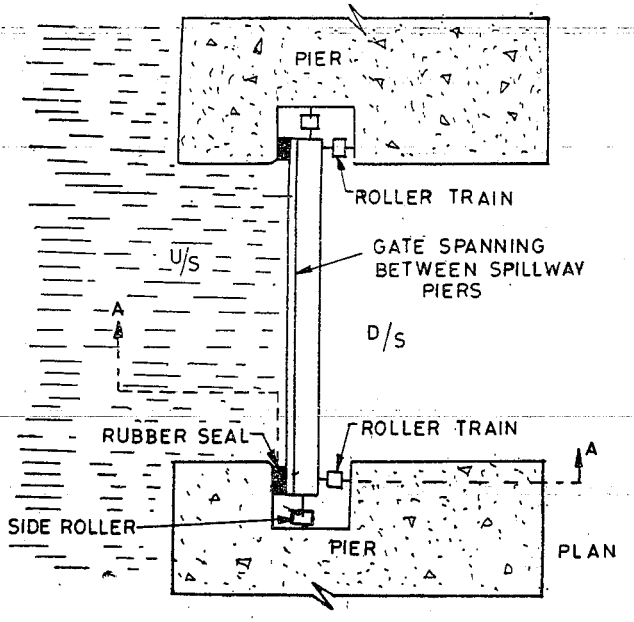
21.17.3. Vertical Lift Gates or Rectangular Gates. These are rectangular gates spanning horizontally between the grooves made in the supporting spillway piers (Fig.21.38). The grooves are generally lined with rolled steel channel sections of appropriate size, so as to provide a smooth bearing surface having sufficient bearing strength and are known as *grove guides.* These rectangular gates move between the groove guides, and can be raised or lowered by a hoisting mechanism at the top.

The gates are often made of steel, although they may be made of concrete or wood. They are generally placed vertical, although they may be kept slightly inclined downstream.

Because of the hydrostatic force caused by the upstream water standing against the gate, large friction is developed between the gate and the downstream groove guides. Hence, if the gate is in direct contact with the guides, as is there in a *sliding gate*, large friction will be developed, and it will be very difficult to move the gate. Hence, in a *sliding gate* relatively larger hoisting capacity is required to operate the gate because of the sliding friction that has to be overcome. The sliding gates are, therefore, seldom used.



(*a*)



(*b*)

Fig. 21.38. 'Vertical stoney gate' or 'Free roller gate'.

This friction problem can be solved by placing cylindrical rollers between the bearing surfaces of the gate and the guide grooves. A train of rollers or wheels is, therefore, generally placed between the gate and the d/s guide, so that the sliding friction is much smaller. These rollers may be placed independent of the gate and the guide, thus eliminating axle friction, as shown in Fig. 21.38. Such an arrangement, when the rollers are neither attached to the gate nor to the guide grooves but rolls vertically between the two when the gate is moved, is known as a *Stoney gate* or a *Free Roller gate.* The design and construction of such a gate is difficult and rollers are, therefore, generally attached to the gate. Such an arrangement in which the rollers or wheels are attached to the gate and ride in tracks on the downstream side of the groove guide, is known as a *Fixed wheels gate* or *Fixed roller type gate.* Rubber seals are used to seal the openings between the upstream leaf plate and the sides of the pier grooves, as shown.

Large vertical lift gates may be counter balanced by a counter weight beam, which is loaded to balance the self-weight of the gate. Hence, hoisting force is required only to balance the frictional resistance.

Vertical lift gates have been used in size 15 m × 15 m (height and span). If the gate height is larger, head room is required for lifting the gate clear of the maximum reservoir level ; thus increasing the height of the operating platform. To reduce the height of the operating platform, high gates may be broken up into two horizontal sections, so that the upper portion may be lifted and removed from the guides before the lower portion is moved. This also reduces the load on the hoisting mechanism.

The discharge through a partially raised vertical gate, takes place by an undershot orifice flow, and the discharge formula is given by $C_d A \sqrt{2g \cdot H_1}$, where A is the area of the opening and H_1 is the water head above the centre line of the opening.

21.17.4. Radial Gates or Tainter Gates. A radial gate has a curved water supporting face made of steel. The curved water face which is in the shape of a sector of a circle is properly braced by steel frame work which is pivoted on horizontal shafts called *trunnions* or *pins* (Fig. 21.39). The pins are anchored in the downstream portion of the spillway piers. The gate can

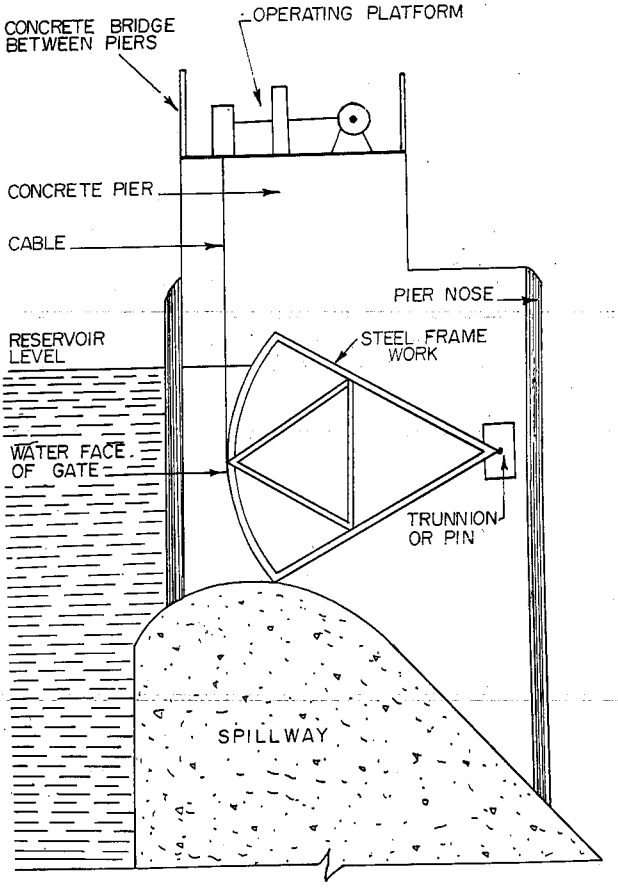


Fig. 21.39. Radial gate or Tainter gate.

thus rotate about the fixed horizontal axis. Hoisting cables are attached to the gate and lead to winches on the hoisting platform. The winches are usually motor driven, although hand driving is possible for smaller works or at times of power failures.

The water-face segment is made concentric to the supporting pins so that the entire water thrust passes through the pins, thus creating no moment against the lifting of the gate. Hence, the lifting force is required only against the weight of the gate, the friction between the seals and the piers, and the frictional resistance at the pins. Counter weights, in order to counter balance the self weight, may also be used, which further reduces the lifting force. Moreover, the hoisting load is nearly constant for all gate openings. Hence, radial gates can be used with smaller lifting force for all heads, and hand operating hoisting mechanism may suffice for smaller works ; whereas in the vertical lift gates of the same size, power mechanism might be needed.

21.17.5. Drum Gates. Drum gates are useful for longer spans of the order of 40 m or so and medium heights say 10 m or so. The drum gate consists of a segment of a cylinder which may be raised above the spillway crest or may be lowered into the *recess* made into the top of spillway.

The U.S.B.R. drum gate (Fig. 21.40) is completely enclosed and is hinged at the upstream end. The buoyant forces due to head water pressure underneath the drum, aid

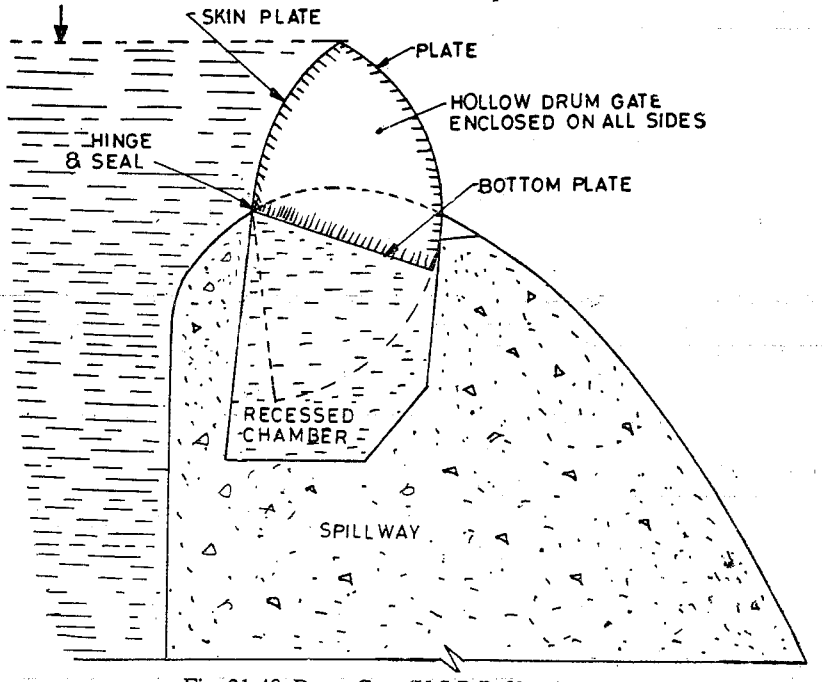


Fig. 21.40. Drum Gate (U.S.B.R. Type).

in its lifting. In this type of dam gate, the drum is enclosed on all the three sides as well as on the ends, thus forming a water tight vessel. When the drum is lowered, it fits into the recess in such a way that the surface becomes coincident with the designed ogee shape of the crest.

The other type of drum gate may have no bottom plate and shall be raised only by the buoyant action of water entering the recess, underneath the skin plate of drum.

The drum gates require large recess and hence, are not suitable for smaller spillways.

PROBLEMS

1. (*a*) "A spillway is a safety valve in a dam". Discuss the statement.

(*b*) Discuss the location of the 'main spillway' and 'subsidiary spillway' in gravity dams as well as in earthen dams.

(*c*) Briefly describe an 'ogee spillway'.

2. (*a*) What are spillways and where are they provided ?

(*b*) Enumerate the various types of spillways, and describe in details the most widely used type.

3. (*a*) What are the different kinds of spillways and how are they selected for individual conditions ?
(Madras University, 1975)

(*b*) Sketch an ogee profile and mark in it the different zones. *(Madras University, 1976)*

4. (*a*) What are spillways and what is their necessity ?

(*b*) Enumerate the different type of spillways which are used in dam construction.

(*c*) Discuss briefly the design principles that are involved in the design of an ogee spillway ; and a chute spillway.

5. (*a*) What is an ogee spillway and how is it designed ?

(*b*) Discuss the necessity of the aeration arrangements that are required in gated ogee spillways.

6. Write down an equation for calculating the discharging capacity of an ogee spillway. How does 'coefficient of discharge' in this equation varies with the :

(*i*) Depth of approach ;

(*ii*) Slope of upstream face of the ogee spillway ;

(*iii*) Submergence of the spillway by the tail water ;

(*iv*) The ratio of design head to actual head over the spillway.

7. (*a*) How would your compute the discharge passing over an ogee spillway. Discuss the various factors affecting the coefficient of discharge in the discharge equation.

(*b*) Compute the discharge over an ogee spillway with coefficient of discharge $c = 2.3$ at a head of 3.8 m. The effective length of the spillway is 110 m. Neglect the velocity of approach.

[**Ans.** 1872 cumecs]

8. Design a suitable section for the overflow section of a concrete gravity dam having the d/s face sloping at a slope of 0.7 H : 1 V. The design discharge for the spillway is 6000 cumecs. The height of the spillway above the river bed is 60 m. The effective length of spillway may be taken as 50 m.

[**Hint.** Follow example 21.1].

9. (*a*) What is meant by a 'spillway' and what is its necessity in dam construction ?

(*b*) Enumerate the different types of spillways, and draw neat sketches for all the types, showing the different parts of each.

10. (*a*) What is a siphon spillway ? Enumerate the two types of saddle syphon spillways, and describe with a neat sketch the component parts and functioning of one of these two types :

(*b*) A siphon spillway has the following cross-section at its throat :

Height of the throat = 1.5 m

Width of the throat = 4 m.

At the design flow, the tail water elevation in 7 m below the summit of the siphon, and the head water elevation is 2 m above the summit, (*i*) Taking a coefficient of discharge as 0.6 ; determine the capacity of the syphon (*ii*) Determine the head that would be required on an ogee spillway 3.8 m long to discharge this flow, if coefficient of discharge is 2.25 ; (*iii*) What length of this ogee weir would be required to discharge the same flow with a head of 2.2 m on the crest.

[**Solution.** (*a*) Discharge through siphon spillway (*i.e.* capacity)

$$= 0.6\,(1.5 \times 4)\sqrt{2 \times 9.81 \times (7 + 2)}$$

$$= 47.8 \text{ cumecs. } \textbf{Ans.}$$

(*b*) $$47.8 = 2.25 \times 3.8\, H^{3/2}$$

or $$H = 3.15 \text{ m. } \textbf{Ans.}$$

(*c*) $4.78 = 2.25\, L\, (2.2)^{3/2}$

or $L = 6.5$ m. **Ans.]**

11. (*a*) Describe briefly the construction and functioning of a 'Hooded type' of siphon spillway. What is its discharge equation ?

(*b*) What is meant by priming ? Discuss the priming arrangements used in saddle siphon spillways.

12. (*a*) What is a 'chute spillway' ? Where is it preferred to ogee and other types of spillways ?

(*b*) Discuss briefly the component parts and their design for a chute spillway.

13. Discuss the design of the following components of a chute spillway :

(*i*) Approach channel

(*ii*) Control structure

(*iii*) Discharge carrier

(*iv*) Energy dissipation arrangement at the bottom.

14. Write short notes on :

(*i*) Volute siphon

(*ii*) Side channel spillway or Chute spillway or Shaft spillway

(*iii*) Ogee spillway

(*iv*) Priming devices for siphon spillways.

15. What is meant by an 'energy dissipator' ? Discuss the various methods used for energy dissipation below spillways.

16. Discuss briefly the various types of energy dissipators that are used for energy dissipation below overflow spillways, under different relative positions of T.W.C. and J.H.C.

17. (*a*) What is a 'spillway gate' and what are the merits and demerits of installing such gates ?

(*b*) Enumerate the important types of spillway gates. Describe with a neat sketch the construction and working of 'Tainter gate'.

18. (*a*) Discuss the merits and demerits of different types of spillway gates.

(*b*) Discuss with a neat sketch the construction, and functioning of a 'Stoney gate' as well as a 'Drum gate'.

19. Write detailed notes on the following :

(*i*) Energy dissipation below spillways

or

Spillway buckets and stilling basins.

(*ii*) Spillway gates.

22

Outlet Works Through Dams and River Intakes

22.1. Sluicesways or Dam Outlets

The most of the water which is stored in a reservoir for irrigation, water supply or power generation purposes, is stored below the spillway crest level. The spillway is provided at normal pool level, such that the floods are discharged safely above the spillway. But, in order to draw water from the reservoir as and when needed, for irrigation, water supply, navigation or power purposes, it is absolutely necessary that outlet works are provided either through the body of the dam or adjacent to it through some hill side at one end of the dam. This water may be discharged into the downstream channel below the dam or may be transported at distances where required (to some power house, etc.) through pipes or canals. The opening, *i.e.* a pipe or a tunnel provided for this withdrawal of water is known as a *dam outlet or a sluiceway.*

The outlets of most of the dams consist of one or more sluiceways with their inlets at about minimum pool level. In most of the cases, a number of outlets are generally provided at different levels ; as a single large capacity outlet may be structurally unsatisfactory or difficult to construct. Moreover, by having more number of smaller capacity outlets, greater control on discharge can be obtained, which can be varied as and when desired. Hence, when wider fluctuations in the demand are anticipated, it is always, advisable to go in for more number of small capacity sluiceways, although it may prove to be somewhat costlier as compared to a few large capacity sluiceways.

A sluiceway is a pipe or a tunnel, circular or rectangular in section, that passes through the body of the dam or through some hillside at one end of the dam and discharges into the stream below. For masonry or concrete gravity dams, these sluiceways can easily pass through the body of the dam or the spillway (Fig. 22.1) ; but for earthen dams, it is preferred to place them outside the limits of the embankments. But, if no such adjacent hill site is available and there is no alternative left but to pass the sluiceway through the dam, *projecting collars* must be provided so as to reduce seepage along the outside of the conduit as shown in Fig. 22.2. The seepage is thus, reduced by increasing the length of the seepage path by at least 25 per cent.

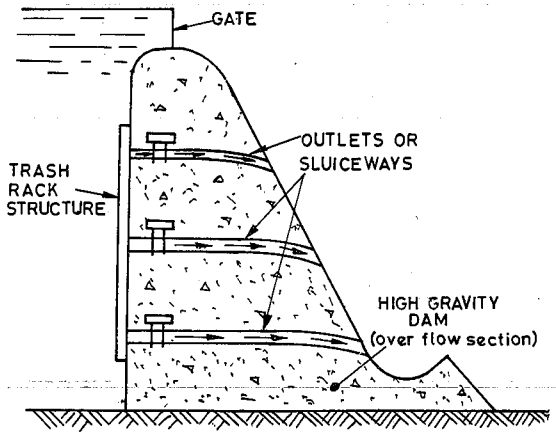


Fig. 22.1. Typical outlet arrangements through a concrete gravity dam.

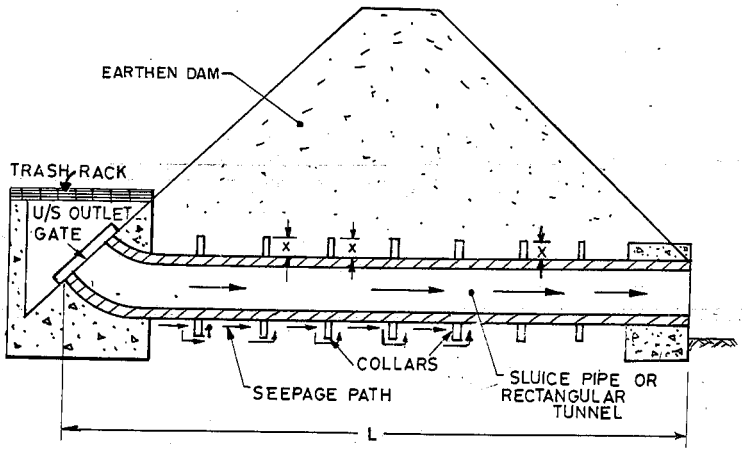


Fig. 22.2. Typical outlet arrangement through an earthen dam.

In Fig. 22.2, if L is the total length of sluiceway from upstream to downstream, the length of the seepage path will be given as :

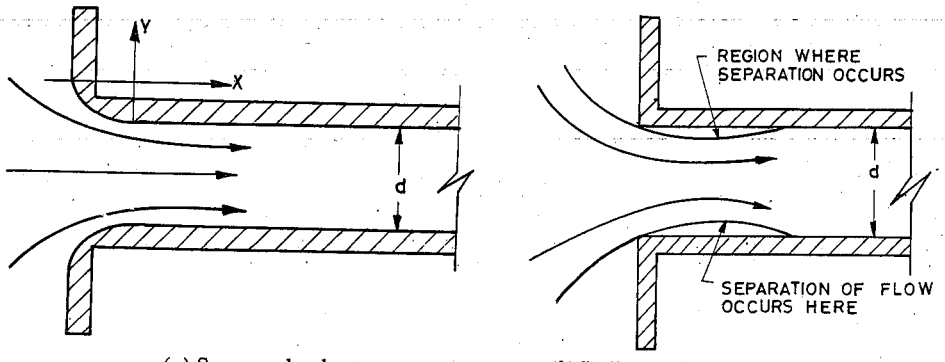
$$= L + (2x)N$$

where x is the projection of each collar ; and

N is the number of collars.

The increase in seepage path, *i.e.* $2 \cdot N \cdot x$ must generally be greater than $L/4$.

The interior of sluiceway tunnel or pipe, must be smooth, without any projections or cavities. The projections or cavities, etc. if at all present, may cause separation of flow from the boundary of the conduit, causing development of negative pressures and the consequent danger of cavitation*. The entrance of the sluiceway is also very impor-



(*a*) Square edged entrance (*b*) Bell-mouthed entrance.

Fig. 22.3

tant, since a square edged entrance [Fig. 22.3 (*a*)] is likely to cause more separation of flow and more danger of consequent cavitation as compared to a bell mouthed or some other type of entrance. There have been several such cases where large scale damages have occurred due to cavitation in the vicinity of square-edged entrances. A bell-mouthed entrance [Fig. 22.3 (*b*)] is much better and superior to any other type, and the

* Explained earlier in Chapter 21.

extra cost involved in shaping this entrance is usually justified except for small projects under low heads.

The shape of the bell mouth is generally elliptical, and Douma has suggested the following equations for it :

(*i*) *For circular conduits.*

$$4X^2 + 44.4\,Y^2 = d^2 \qquad ...(22.1)$$

where X, Y are the co-ordinates of any point on the curve as defined in Fig. 22.2 (*b*) ; and

d is diameter of the circular conduit.

(*ii*) *For rectangular tunnels or conduits*

$$X^2 + 10.4\,Y^2 = d^2 \qquad ...(22.2)$$

where X and Y have same meaning as above ; and

d is the width or height of the conduit, depending on whether the sides or top and bottom curves are being designed.

22.2. Hydraulics of Outlet Works

The discharge passing through the dam outlet can be calculated easily, be using the equation :

$$Q = C_d \cdot A\sqrt{2gH} \qquad ...[22.3(a)]$$

where Q = Discharge

A = Area of outlet sluice

H = Differential head causing flow, *i.e.* the difference of u/s and d/s water levels.

C_d = Coefficient of discharge, whose value depends upon various factors such as the type of gate and trash rack provided, conduit friction, transitions etc.

For precise designs, the total head losses encountered in the outlet conduits may be calculated. These losses include those caused by trash racks, conduit entrance, conduit friction, gates and valves, transitions and bends. The entrance loss may be taken as $0.5\,\frac{V^2}{2g}$ for square edged entrance, and equal to $0.04\,\frac{V^2}{2g}$ for bell mouthed entrance, where V is the flow velocity through the conduit. Head losses caused by conduit friction may be calculated by standard pipe formulas, such as :

$$h_L = \frac{f' \cdot LV^2}{2gd}$$

or

$$h_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}}$$

The head loss through the gate depends upon the type of gate and valve provided. A loss of about $0.2\,\frac{V^2}{2g}$ may be taken for fully open gate and butterfly valves. This head loss is taken as nil for ring follower gates. The head loss through the trash rack depends upon the velocity, and may be taken as shown in Table 22.1.

Table 22.1. Head Loss through Trash Racks

Velocity through trash rack in m/sec	*Head loss in metres*
0.15	0.006
0.30	0.03
0.45	0.09
0.62	0.15

The net effective head which is responsible for flow, should then be taken as $= H_{eff}$ = Differential head (H) – Head losses. The discharge can then be calculated easily, using

$$Q = A\sqrt{2g \cdot H_{eff}} \qquad ...[22.3\ (b)]$$

22.3. River Intakes

When water is withdrawn through a conduit from a river or a reservoir independently, and as such the entrance of the conduit is not an integral part of the dam or any other related structure, then an intake structure must be constructed at the entrance of the conduit. An intake structure may vary from a simple concrete block supporting the end of the conduit pipe to huge concrete towers, depending upon various factors such as, reservoir characteristics, capacity and discharge requirements, climatic conditions, etc. The basic function of an intake structure is to help in safely withdrawing water from the reservoir over a predetermined range of pool levels, and thus to protect the conduit from being damaged or clogged by ice, trash, debris, waves, etc.

Various types of intake structures are discussed below :

(1) **Simple Submerged Intakes.** A simple submerged intake consists of a simple concrete block, or a rock filled timber crib supporting the starting end of the withdrawal pipe, as shown in Figs. 22.4 (*a*) and (*b*).

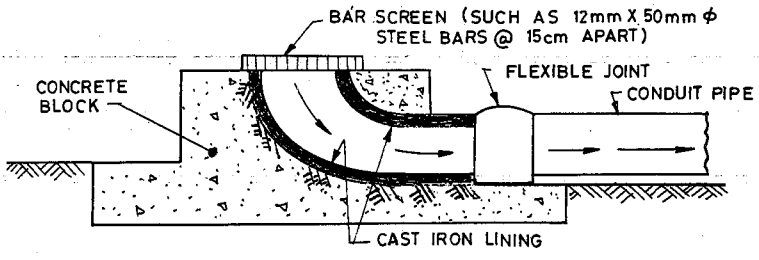


Fig. 22.4. (*a*) Simple concrete block submerged intake.

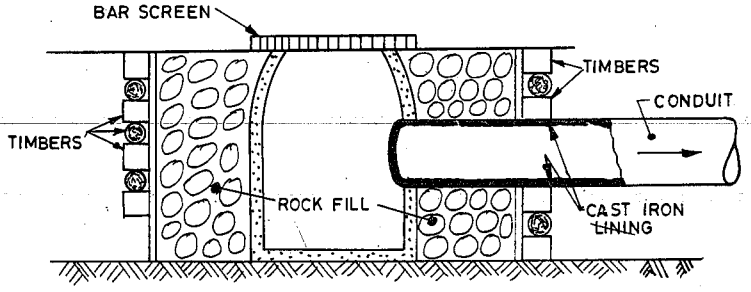


Fig. 22.4. (*b*) Rock-filled timber-crib submerged intake.

Such intake structures should be placed in the river or the reservoir at a place where they may not get buried under the sediment. These submerged intakes are very economi-

cal and donot obstruct navigation, and are, therefore, widely used on small works, and are particularly suitable as water-supply intakes from small rivers*. They are sometimes used as intakes to sluiceways of earthen dams with hydraulically operated gates for flow regulation. These intakes are not used on bigger projects as their main disadvantage is the fact that they are not easily accessible for repairing of their gates, etc.

(2) **Intake Towers.** Intake towers are generally used on large projects and where there are large fluctuations of water level. Openings at various levels, called *ports* are generally provided in these concrete towers, which may help in regulating the flow through the towers and permit some selection of the quality of water to be withdrawn. If the ports are submerged at all levels, then there is no problem of any clogging or damage by ice or debris, etc. However, the level of the lowest port should be high enough above the reservoir bed, so that the sediment is not drawn into them. There are two major types of intake towers, *viz.*

(*i*) Wet intake towers ; and

(*ii*) Dry intake towers.

(*i*) *Wet intake towers.* A typical section of a wet intake tower is shown in Fig. 22.5 (*a*). It consists of a concrete circular shell filled with water upto the reservoir level, and has

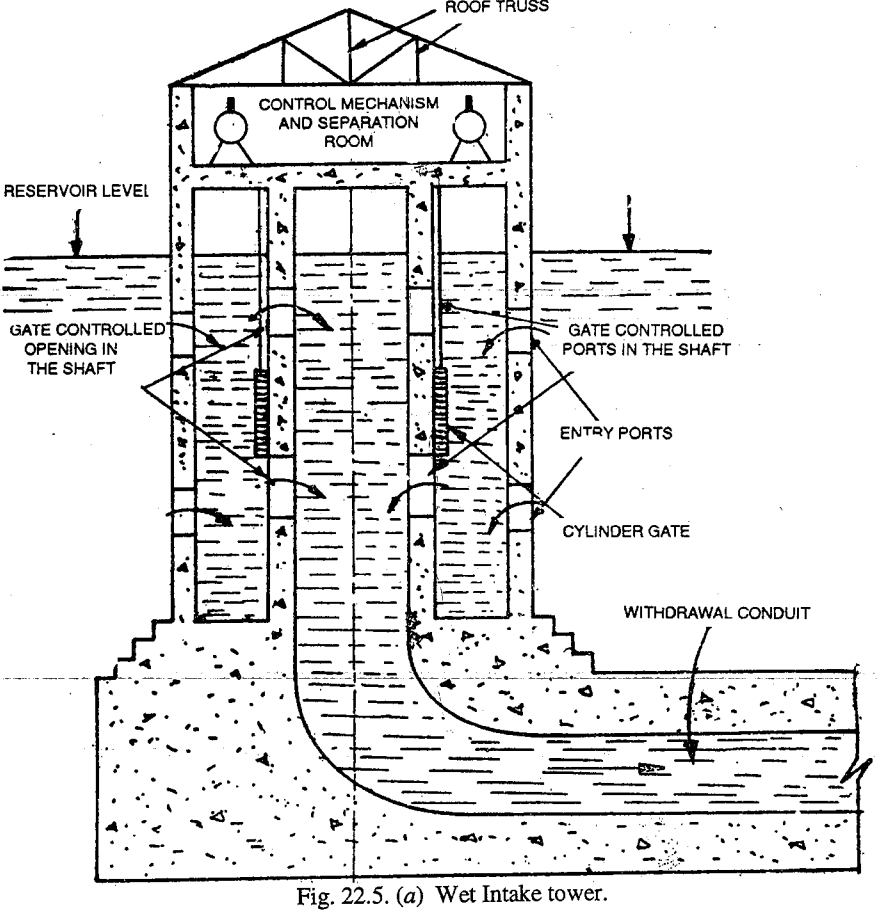


Fig. 22.5. (*a*) Wet Intake tower.

* More detailed description of river and canal intakes is given in "Water Supply Engineering" by the same author.

a vertical inside shaft which is connected to the withdrawal pipe. Openings are made into the inside shaft also. Gates are usually placed on the shaft, so as to control the flow of water into the shaft and the withdrawal conduit.

(ii) Dry Intake Towers. The essential difference between a wet intake tower and a dry intake tower is that, whereas in a wet intake tower, the water enters from the entry ports into the tower and then it enters into the conduit pipe through separate gate controlled openings ; in a dry intake tower, the water is directly drawn into the withdrawal conduit through the gated entry ports as shown in Fig. 22.5 (*b*). *A dry intake tower will, therefore, have no water inside* the tower if its gates are closed, whereas the

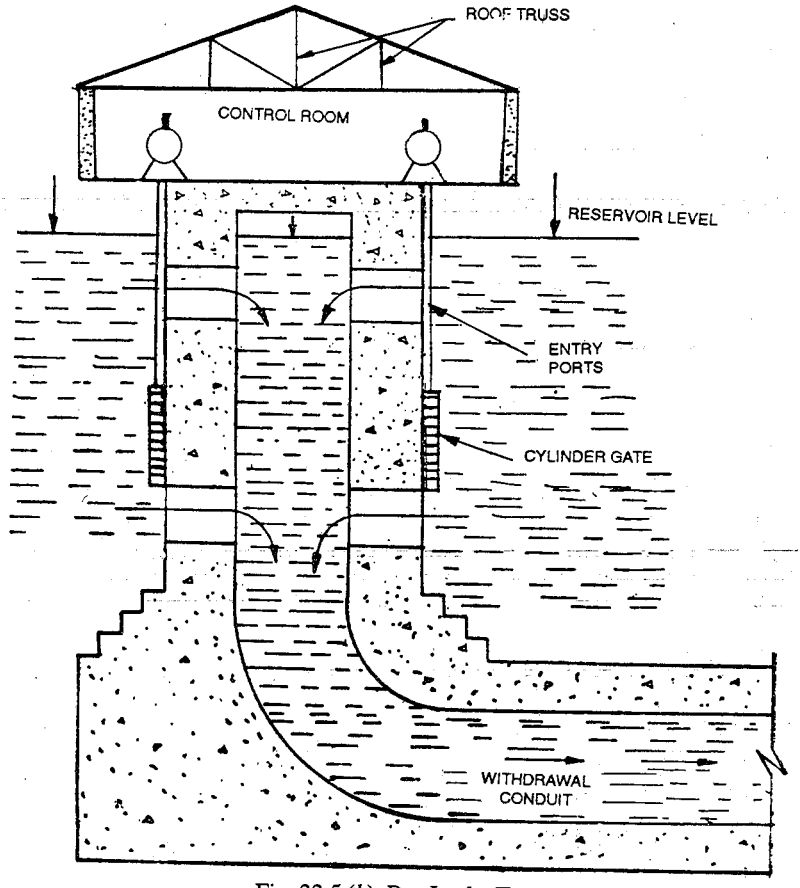


Fig. 22.5 (*b*). Dry Intake Tower.

wet intake tower will be *full of water even if its gates are closed.* When the entry ports are closed, a dry intake tower will be subjected to additional buoyant forces and hence, must be of heavier construction than the wet intake towers. However, the dry intake towers are useful and beneficial in the sense that water can be withdrawn from any selected level of the reservoir by opening the port at that level.

Intake towers are huge structures standing in the river, and hence should be located so as not to interfere with navigation, and must be properly designed so as to withstand the worst possible combination of various forces, such as hydrostatic pressures, wind and earthquake forces, and forces caused by waves, ice and debris, etc.

22.4. Trash Racks

The entrances to intakes and dam outlets are generally covered with trash racks so as to prevent the entry of debris, ice, etc. into the conduit. These racks are generally nothing but bar screens, made from steel bars spaced at 5 to 15 cm centre to centre (in both directions) depending upon the maximum size of the debris required to be excluded from entering the conduit. The velocity of flow through the trash rack is kept low (generally less than 0.62 m/sec), so as to minimise losses. This is sometimes accomplished by constructing the trash rack in the form of a half cylinder, as shown in Fig. 22.6.

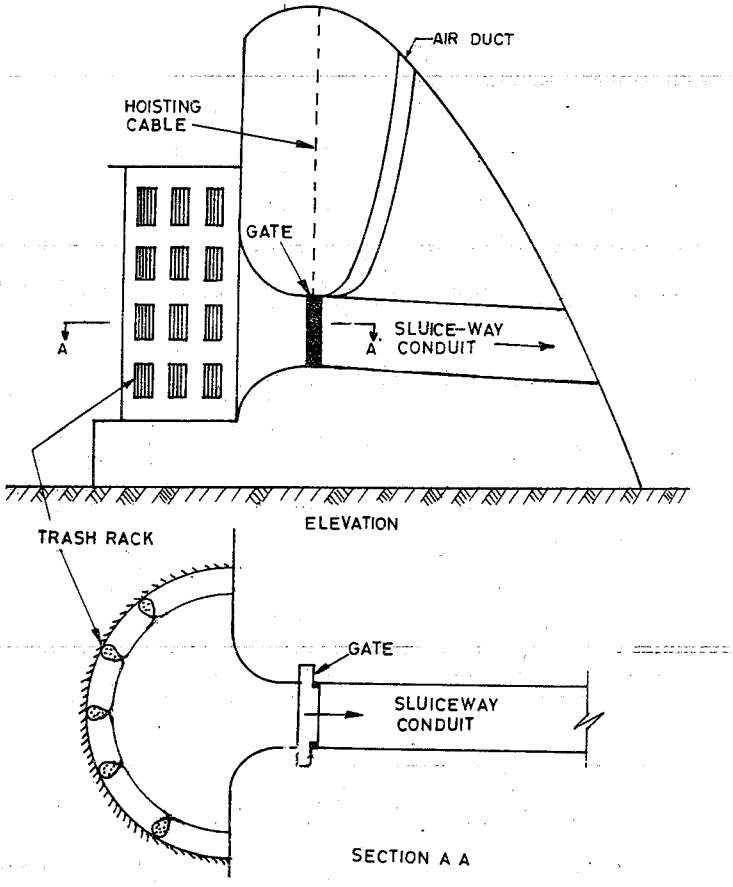


Fig. 22.6. Half Cylinder-Trash rack and Tractor gate installation.

The floating debris, ice, etc. which are stopped by the racks and thus get collected on them, are removed by manual labour when required ; but on large projects, where much debris are expected, automatic power-driven racks are used. Steam or electrically heated racks are preferred in cold countries, so as to prevent ice formation on trash racks.

PROBLEMS

1. (*a*) What is meant by a 'Dam sluice' ? Why are such sluices necessary in dam construction ?

(*b*) Show with neat sketches, the provision of sluiceways in

(*i*) an earth dam.

(*ii*) a concrete gravity dam.

2. Write short notes on :

(*i*) Sluiceways in dams.

(*ii*) River intakes.

(*iii*) Trash Racks.

23

Pressure Conduits

23.1. Definition, Etc.

A pressure conduit (such as a penstock) is a pipe which runs under pressure and, therefore, runs full. This type of conduits prove economical than canals or flumes, because they can generally follow shorter routes. Moreover, their biggest advantage is: that the water or any other fluid flowing through them is not exposed anywhere and hence, there are no chances or very less chances of its getting polluted. Hence, these pressure conduits are preferably used for city water supplies. Since the water wasted in percolation, evaporation, etc. is also saved, when water is carried through these conduits, they are preferably used when water is scarce. The flow of water through conduit pipes is generally turbulent, and hence, it will be considered so, while dealing with the hydraulics of flow through such pipes.

23.2. Hydraulics of Flow and Discharging Capacities of Pressure Conduits

The pressure conduits are designed in such a way that the available pressure head is just lost in overcoming the frictional resistance offered to the flow of water by the pipe. The velocities generated in these pipes should neither be too small to require a large size diameter pipe, nor should be so large as to cause excessive loss of pressure head.

The head loss by pipe friction may be found by using either of the following formulas :

(*i*) **Darcy-Weisbach Equation.** It states that

$$H_L = \frac{f' \cdot L \cdot V^2}{2gd}$$

where H_L = Head loss in metres.

L = Length of the pipe in metres.

d = Diameter of the pipe in metres.

V = Velocity of flow through the pipe in m/sec.

f' = The friction factor, which depends upon Reynold number $\left(i.e.\ R_e = \frac{Vd}{\nu}\right)$ and relative roughness of the pipe.

The relative roughness $\left(\delta = \frac{2e}{d}\right)$ of a pipe depends on the absolute roughness (e) of the inside surface and the diameter of the pipe d. The approximate values of f' are given by the following empirical relations :

$$f' = 0.04\left[1 + \frac{1}{35\,d}\right] \text{ for old pipes} \qquad ...(23.2)$$

and $$f' = 0.02\left[1 + \frac{1}{35\,d}\right] \text{ for new pipes} \quad ...(23.3)$$

The accurate value of f' depends on R_e and δ, and may be given by formula, such as

$$f' = a + \frac{b}{R_e^m} \quad ...(23.4)$$

where a, b and m are constants depending upon δ.

For R_e varying between 20,000 to 20,00,000, Schiller has given

$$f' = 0.005 + \frac{0.396}{R_e^{0.3}} \quad ...(23.5)$$

and Nikuradse has given for values of R_e varying between 20,000 to 32,40,000 as

$$f' = 0.0032 + \frac{0.221}{R_e^{0.237}} \quad ...(23.6)$$

(*ii*) **Manning's formula.** Manning's formula is also applicable to turbulent flow in pressure conduits and yields good results, provided the roughness coefficient n is accurately estimated. Head loss according to Manning's equation is given as :

$$H_L = \frac{n^2 \cdot V^2 \cdot L}{R^{4/3}} \quad ...(23.7)$$

where n = Manning's rugosity coefficient.

L = Length of pipe in metres.

V = Flow velocity through pipe in m/sec.

R = Hydraulic mean depth of pipe

$$\left[\text{i.e., } R = \frac{A}{P} = \frac{\frac{\pi d^2}{4}}{\pi d} = \frac{d}{4}\right] \text{ in metres}$$

(*iii*) **Hazen-William's formula.** This formula is widely used for pipe flows and states

$$V = 0.85\, C_H \cdot R^{0.63} S^{0.54} \quad ...(23.8)$$

where C_H = Coefficient given by Table 23.1.

R = Hydraulic mean depth of pipe, $= (d/4)$ in metres.

S = Slope of the energy line.

V = Flow velocity through the pipe in m/sec.

Table 23.1. Values of C_H for Hazen William's Formula

Pipe material	*Value of C_H*
Concrete (regardless of age)	130
Cast Iron	
New	130
5 years old	120
20 years old	100
Welded steel (New)	120
Riveted steel (New)	110
Vitrified clay	110
Brick Sewers	100
Asbestos-cement	140

The carrying capacities of pipes decrease with time, because with the passage of time, their interior surfaces go on becoming more and more rough. Hence, it is customary to select higher values of e and n and lower values of C_H for old pipes than are generally taken for new pipes.

23.3. Forces Acting on Pressure Conduits

Pressure pipes must be designed to withstand the following forces :

(1) *Internal pressure of water*

(2) *Water hammer pressures.*

(3) *Pressure due to external loads* (when buried under the ground).

(4) *Temperatures stresses* (when laid above the ground).

(5) *Longitudinal stresses* due to flow around bends or change in cross-section.

(6) *Flexural stresses* (when laid over supports at intervals or on bridges).

These forces are discussed below :

(1) **Internal Pressure of Water.** The pressure exerted on the walls of the pipe by the flowing water, in the form of Hoope's tension, is the internal pressure. The circumferential tensile stress produced is given as :

$$\sigma_1 = \frac{p_1 d}{2t} \text{ in kN/m}^2 \qquad ...(23.9)$$

where p_1 = Internal static pressure in kN/m^2.

d = Diameter of the pipe in metres.

t = Thickness of the pipe shell in metres.

σ_1 = Circumferential tensile stress to be counteracted by providing Hoope's reinforcement.

(2) **Water Hammer Pressure.** When a liquid flowing in a pipe line is abruptly stopped by the closing of a valve, the velocity of the water column behind, is retarded, and its momentum is destroyed. This exerts a thrust on the valve and additional pressure on the pipe shell behind. The more rapid the closure of the valve, the more rapid is the change in momentum, and hence, greater is the additional pressure developed. The pressures so developed are known as *water-hammer pressures* and may be so high as to cause bursting of the pipe shell (due to increased circumferential tension) if not accounted for in the designs.

The maximum pressure developed in pipe lines due to water hammer is given by the formula

$$p_2 = \frac{14.762 \cdot V}{\sqrt{1 + \frac{K \cdot d}{t}}} \qquad ...(23.10)$$

where V = Velocity of water just before the closing of the valve in m/sec.

d = Diameter of pipe in metres.

t = Thickness of pipe shell in metres.

K = Constant

$$= \frac{\text{Modulus of elasticity of pipe material}}{\text{Bulk modulus of elasticity of water}}$$

The value of K for steel comes out to be 0.01, for cast iron = 0.02, and for cement concrete = 0.1.

The water hammer pressure can be calculated by using the above equation. But for design purposes, its value is generally taken as 840 kN/m^2 for small size pipes of 7.5 to 25 cm diameter, and is gradually reduced to 490 kN/m^2 for pipes above 1.2 m diameter, as given in Table 23.2.

Table 23.2. Water Hammer Pressures

Pipe diameter in cm	*Water hammer Pressure in kN/m^2*
7.5 to 25	840
30 to 40	770
50	630
60	600
75	560
90	490
105 to 135	490

The circumferential tensile stress caused by water hammer pressure may then be calculated easily by using

$$\sigma_2 = \frac{p_2 \cdot d}{2t}$$

(3) **Stress due to External Loads.** When large pipes are buried deep under the ground, the weight of the earth-fill may produce large stresses in the pipe material. The stress due to the external earth fill load is given by

$$f = 22.7 \frac{h \cdot d^2}{t} \qquad ...(23.11)$$

where h = depth of the earth-fill above the crown in metres.

d = diameter of pipe in metres.

f = stress produced in kN/m^2.

Note. In the above formula, it is assumed that the earth to the sides does not give any lateral support and weighs about 18.4 kN/m^3.

(4) **Temperatures Stresses.** When pipes are laid above the ground, they are exposed to the atmosphere and are, therefore, subjected to temperature changes. They expand during day time and contract at night. If this expansion or contraction is prevented due to fixation or friction over the supports, longitudinal stresses are produced in the pipe material. The amount of these stresses may be calculated by the formula :

$$f = E \cdot \alpha \cdot T \qquad ...(23.12)$$

where E = Modulus of elasticity of the pipe material.

α = Co-efficient of expansion of the pipe material.

T = Change in temperature in °C.

Expansion joints at suitable intervals (say 20 m to 30 m or so) must be provided to counteract these stresses.

(5) **Stresses due to Flow Around Bends and Change in Cross-Section.** Whenever the velocity of a flow (either magnitude or direction) changes, there is a change in the momentum, and therefore, by Newton's Second Law, a force is exerted, which is proportional to the rate of change of momentum. The force required to bring this change in momentum comes from the pressure variation within the fluid and from forces transmitted to the fluid from the pipe walls.

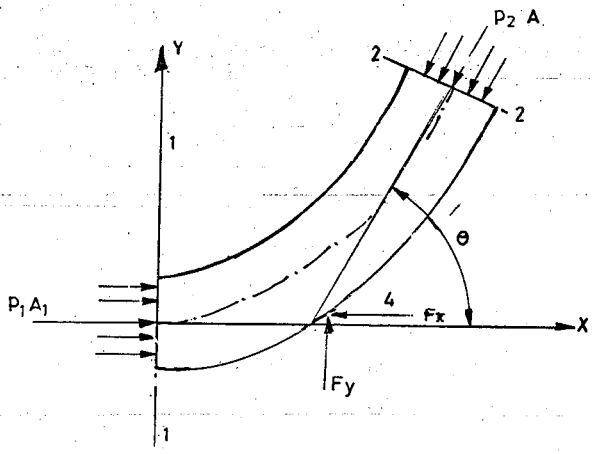


Fig. 23.1. Forces at a pipe bend.

The free-body diagram of various forces acting on the water contained in a horizontal pipe bend is shown in Fig. 23.1. Applying momentum equation, and resolving the forces in x and y direction, we get

$$p_1A_1 - F_x - p_2A_2 \cos\theta = \frac{\gamma_w Q}{g}\left[V_2 \cos\theta - V_1\right] \qquad ...(23.13)$$

and

$$F_y - p_2A_2 \sin\theta = \frac{\gamma_w Q}{g}\left[V_2 \sin\theta\right] \qquad ...(23.14)$$

where p_1 and p_2 are the pressures, V_1 and V_2 are the velocities at sections 1-1 and 2-2 respectively. The forces F_x and F_y are the forces which are transmitted from the pipe to the water. An equal and opposite force must, therefore, be developed in the form of stresses in the pipe wall. Similar forces will be developed when the cross-sectional area of the pipe is suddenly changed, as shown in Fig. 23.2.

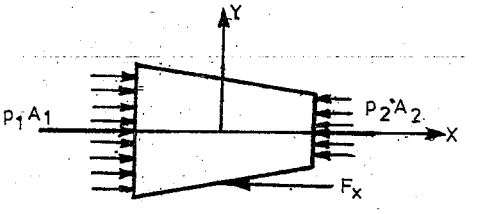


Fig. 23.2. Forces at a change in X-section of a pipe.

Due to these impressed external forces or stresses, the pipe line may be thrown out of alignment as and when such situations arise, unless held firmly by anchoring it in massive blocks of concrete or stone masonry.

(6) **Flexural Stresses.** Many a times, steel pipes are laid over concrete supports, built above the ground ; and sometimes the rain water, etc. may wash off the ground from below the pipes at intervals. Under all such circumstances, bending stresses get produced in the pipe, since the pipe then acts like a beam with loads resulting from the weight of the pipe, weight of water in the pipe and any other superimposed loads. The stresses caused by this beam action may be determined by usual methods of analysis applied to the beams. However, these stresses are generally negligible except for long spans or where there are huge superimposed loads.

23.4. Various Types of Pressure Conduits

Depending upon the construction material, the various types of pressure conduits are :

(1) Cast iron conduits.

(2) Steel conduits.

(3) Reinforced cement concrete conduits.

(4) Hume steel conduits.

(5) Vitrified clay conduits.

(6) Asbestos cement conduits.

(7) Miscellaneous types of conduits.

The selection of a particular type of material for the conduit depends mainly upon their relative economy and also upon the availability of material, labour, etc. for construction in the vicinity of site.

(1) **Cast Iron Conduits.** Cast iron pipes are widely used for city water supplies. They are sufficiently resistant to corrosion and may last as long as 100 years or so. They are generally manufactured in lengths of about 3.6 metres but may be manufactured in lengths up to 6 m or so on special orders. Cast iron pipes in different thicknesses, are generally available for withstanding different pressures upto a maximum of 2400 kN/m^2 (24.5 kg/cm^2).

In order to protect the pipes from corrosion, almost all the metal pipes are given some kind of protective coatings. Larger pipes are generally protected by bituminous linings or by cement mortar (1 : 2) linings. Smaller size pipes, such as those required for small service lanes in a water supply distribution system, are generally coated with zinc, and are known as galvanised iron pipes.

Bell and spigot joint (Fig. 23.3) is often used for connecting cast iron pipes.

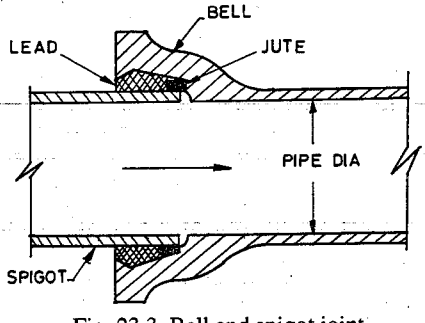


Fig. 23.3. Bell and spigot joint for cast iron pipes.

A few strands of jute are wrapped around the spigot before inserting it into the bell, and then, more jute is packed into the joint. The remaining space between the bell and the spigot is finally filled with molten lead, which gets solidified and tightly chalked into the joint after cooling, and thus making a water-tight joint. The quantity of lead required varies from 3.5 to 4 kg for 15 cm dia pipe to about 45 to 50 kg for 1.2 metres dia pipe.

(2) **Steel conduits.** Steel pipes can be used for large diameters, say upto or above 6 metres or so. Steel plates of varying thicknesses for withstanding different pressures, are generally bent and welded or sometimes riveted. Welded pipes are smoother and stronger than riveted pipes, and are, therefore, generally used these days. Since steel is strong in tension, even large size diameter pipes can be made of thin shells. They are, therefore, lighter than cast iron pipes.

Galvanised steel pipes with circumferential corrugations are much more stronger than ordinary steel pipes. They are, therefore, used where large dia pipes of smaller wall thicknesses are required. Such pipes are usually manufactured in various sizes varying

from 20 cm to 2 metres in diameter, and are lighter and can be more easily transported at distances.

Expansion joints are generally not required in steel pipes which are buried under the ground, because they are not subjected to large temperature changes. However, pipes which are exposed to the atmosphere may require expansion joints so as to minimise temperature changes.

Steel pipes get rusted quickly, which, reduces their life as well as carrying capacities. They are, therefore, protected on the inside as well as outside by protective coatings. Under ordinary conditions, steel pipes may last as long as 40 years or so. The various pipe lengths in case of steel pipes are connected by welded or riveted joints.

(3) **Reinforced Cement Concrete Conduits.** Ordinary cement concrete pipes are manufactured in small sizes (*i.e.* up to say 0.6 m diameter) ; while they are reinforced with steel for large dia pipes. They are easily available in sizes up to diameters say about 1.8 metres and may be got manufactured for larger diameters say up to about 4.5 metres, on special orders. These pipes may either be prepared at site by transporting various ingradients (*i.e.* cement, steel aggregates, water, etc.) or can be manufactured in factories and then transported to site. They are known as '*cast in situ*' pipes in the former case, and '*precast pipes*' in the latter case. Cast in situ pipes are useful when the site conditions are difficult, and where it may be difficult to carry the pipes. But since such pipes are cast at site, lesser supervision and check is possible, as compared to the case of precast pipes which are cast in the factories, and thus subjected to greater quality control and supervision.

R.C.C. pipes can be manufactured in three different ways, *viz.*

(*a*) pipes having bar and mesh reinforcement, and concrete poured by usual ordinary methods of concrete pouring, and tamped ;

(*b*) pipes made by rotating the mould or the form rapidly about the pipe axis. The mould contains concrete and fabricated reinforcement. The centrifugal force throws off the concrete which then spreads in a uniform layer over the internal surface of the mould and embed the reinforcement ; thus providing a high density watertight concrete surface. This type is known as *centrifugal type* ;

(*c*) the third type of pipes are made by lining thin cylindrical steel shells both internally and externally with rich cement concrete. These are stronger and more water-tight than the first two. They are known as *cylinder type.*

R.C.C. pipes are generally made from 1 : 2 : 4 cement concrete with maximum size of aggregates as 6 mm. They are provided with circumferential reinforcement to carry hoop tension and a nominal longitudinal reinforncement equal to 0.25% of cross-sectional area of concrete. The thickness of R.C.C. pipes generally varies from 7.5 cm to 0.6 m for pipes of diameters* varying from 0.2 m to 2.75 m.

(4) **Hume Steel Conduits.** Hume steel pipes are R.C.C. pipes patented under this name. They consist of thin steel shells coated from inside with cement mortar by centrifugal process. The thickness of the inside coating varies from 1.2 cm to 3 cm depending upon the size of the pipe. They are also coated from outside so as to protect the steel shell from external weather or soil action. The thickness of external coating is 2.5 cm for pipes upto 1 metre in diameter and is 3.75 cm for pipes of larger diameters.

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* The standard mentioned diameter of pipes is always the internal diameter.

The thickness of steel shell depends upon the size of the pipe and also upon the pressure to be borne by the pipe. Like all R.C.C. pipes, they are heavy and difficult to handle.

(5) **Vitrified Clay Conduits.** They are generally not used as pressure conduits, but are extensively used for carrying sewage and drainage at partial depths. These pipes are free from corrosion and provide a smooth hydraulically efficient surface. They are not used as pressure pipes, because clay is weak in tension, and formation of water-tight joints becomes difficult in them. Clay pipes are commonly made in lengths of about 0.6 m to 1.2 m or so.

(6) **Asbestos Conduits.** Asbestos, silica and cement are converted under pressure to a dense, homogeneous material possessing high strength, called *asbestos cement.* This material is used for casting these pipes. The asbestos fibre which is thoroughly mixed with cement serves as reinforcement. These pipes are generally available in different sizes, say from 10 cm to 90 cm in diameters and in about 4 metres lengths. These pipes are generally made in 4 different grades to withstand pressures of 350 kN/m^2 to 140 kN/m^2. These pipes are joined by means of special type of flexible joint called '*simplex joint*'. The assembly consists of a pipe sleeve and two rubber rings which are compressed between the pipe and the interior of the sleeve.

Advantages. The advantages of asbestos cement pipes are :

(*i*) They are light and hence easy to transport.

(*ii*) They can be easily assembled without skilled labour.

(*iii*) They are highly resistant to corrosion.

(*iv*) They are highly flexible and may permit as much as 12° deflection in laying them around curves.

(*v*) Expansion joints are not required as the coefficient of expansion is low and the joints are also flexible.

(*vi*) They are very smooth and thus provide a hydraulically efficient pipe. Their carrying capacities do not reduce with time.

(*vii*) They are very suitable to be used as small size distribution pipes.

Disadvantages and limitations. Their disadvantages are :

(*i*) They are costly.

(*ii*) These pipes do not have much strength and are brittle and soft. They are liable to get damaged by excavating tools or during transportation transits.

(*iii*) The rubber joint seals may deteriorate if exposed to gasoline or other petroleum products, and hence cannot be used for transporting petroleum products.

(7) **Miscellaneous Types of Conduits.** Various other materials which may be used for manufacturing pipe conduits are : copper, wrought iron, plastics, etc. *Copper pipes* are very costly although they are highly resistant to acidic as well as alkaline waters, they can be bent easily and do not sag due to heat. They are, therefore, very useful for carrying hot water in the interior of the buildings. Wrought iron pipes are lighter than cast iron pipes and can be more easily cut, threaded and worked. They are more costly but neat in appearance. They are generally manufactured in small sizes and are very useful for indoor works. However, they corrode quickly and are less durable. they are,

therefore, generally protected by galvanising them with zinc coatings and they are then known as galvanised iron pipes.

Plastic pipes are lighter and free from corrosion. But they are of low strengths and less durable. Moreover, they can not withstand high temperatures exceeding 60°C or so. They may, however, be used for very minor works in house connections, etc. HDPE pipes are however, being increasingly used these days for various uses, such as in tubewell boarings, sewage mains, etc. etc. They are however, still not being made in large sizes suitable as penstock pipes.

PROBLEMS

1. (*a*) What is meant by a pressure conduit ? Discuss briefly the forces acting on a pressure conduit.

(*b*) Write a brief note on the design capacities of pressure pipes.

2. What are the different construction materials which are used for manufacturing pressure pipes ? Discuss their comparative merits and demerits.

3. Write a note on Pressure conduits'.

24

Hydro-electric Power

Faraday had shown that when a coil is rotated in a magnetic field, electricity is generated. Thus, in order to produce electrical energy, it is necessary that we should produce mechanical energy, which can be used to rotate the 'coil'. The mechanical energy is produced by running a prime mover (*known as turbine*) by the energy of fuels or flowing water. This mechanical power is converted into electrical power by electric generator which is directly coupled to the shaft of the turbine, and is thus run by the turbine. The electrical power, which is consequently obtained at the terminals of the generator, is then transmitted to the area where it is to be used for doing work.

The plant or machinery which is required to produce electricity (*i.e.* Prime mover + Electric generator) is collectively known as the *Power plant.* The building, in which the entire machinery along with other auxiliary units is installed, is known as the *Power house.*

24.1. Thermal and Hydropower

As stated earlier, the turbine blades can be made to run by the energy of fuels or flowing water. When fuel is used to produce steam for running the steam turbine, then the power generated is known as *Thermal power.* The fuel which is to be used for generating steam may either be (*i*) an ordinary fuel such as coal, fuel oil, gas, etc., or (*ii*) atomic fuel or nuclear fuel. Coal is simply burnt to produce steam from water and is the simplest and the oldest type of fuel. Diesel oil, etc. may also be used as fuels for producing steam. Atomic fuels such as uranium or thorium may also be used to produce steam. When conventional type of fuels such as coal, oil, etc. (called fossils) are used to produce steam for running the turbines, the power house is generally called an *Ordinary thermal power station* or *Thermal power station.* But when atomic fuel is used to produce steam, the power station, which is essentially a thermal power station, is called an *Atomic power station* or a *Nuclear power station.* In an ordinary thermal power station, steam is produced in a water boiler, while in the atomic power station, the boiler is replaced by a nuclear reactor and steam generator for raising steam. The electric power generated in both these cases is known as *Thermal power* and the scheme is called *Thermal power scheme.*

But, when the energy of the flowing water is used to run the turbines, then the electricity generated is called the *Hydroelectric power.* This scheme is known as *Hydel scheme*, and the power house is known as *Hydel power station* or *Hydroelectric power station.* In a hydel scheme, a certain quantity of water at a certain potential head, is essentially made to flow through the turbines. The head causing flow, runs the turbine blades, and thus producing electricity from the generator coupled to the turbine. In this chapter, we are concerned with hydel schemes only.

24.2. Classification of Hydel Plants

24.2.1. Classification of Hydroplants on the Basis of Hydraulic Characteristics. On the basis of this classification, the hydro plants may be divided into the following types:

(*i*) *Run-off river plants*

(*ii*) *Storage plants*

(*iii*) *Pumped storage plants*

(*iv*) *Tidal plants*

They are described below :

(*i*) **Run-off River Plants.** These plants are those which utilise the minimum flow in a river having no appreciable pondage on its upstream side. A weir or a barrage is sometimes constructed across a river simply to raise and maintain the water level at a pre-determined level within narrow limits of fluctuations, either solely for the power plant or for some other purpose where the power plant may be incidental. Such a scheme is essentially a low head scheme and may be suitable only on a perennial river having sufficient dry wheather flow of such a magnitude as to make the development worth while. [Pl. refer Fig. 24.2]

Run-off river plants generally have a very limited storage capacity, to supplement the normal flow. Such a small storage capacity, called **pondage,** is provided for meeting the hour to hour fluctuations of load or of streamflow over a day ; or occassionally, day to day fluctuations over a weekly cycle. When the available discharge at site is more than the demand (during off-peak hours), the excess water is temporarily stored in the pond on the upstream side of the barrage, which is then utilised during the peak hours.

The power stations constructed on diversion canals (Irrigation & Power Canals) called *Diversion canal plants*, can also be placed in this category.

The various examples of run-off river plants are : Ganguwal and Kotla power houses located on Nangal Hydel Channel, Mohammad Pur and Pathri power houses on Ganga Canal, and Sarda power house on Sarda Canal.

(*ii*) **Storage Plants.** A storage plant is essentially having an upstream storage reservoir of sufficient size, so as to permit sufficient carry-over storage from the monsoon season to the dry summer season, and thus to develop a firm flow substantially more than the minimum natural flow. In this scheme, a dam is constructed across the river, and the power house may be located at the foot of the dam such as in Bhakra, Hirakund, Rihand projects, etc. The power house may sometimes be located much away from the dam (on the down-stream side). In such a case, the power house is located at the end of tunnels which carry water from the reservoir. The tunnels are connected to the power house machines by means of pressure penstocks which may either be underground (as in Maithon and Koyna projects) or may be kept exposed (as in Kundah project).

When the power house is located near the dam, as is generally done in the low head installations ; it is known as *Concentrated fall hydroelectric development*. But when the water is carried to the power house at a considerable distance from the dam through a canal, tunnel, or penstock ; it is known as a *Divided fall development*.

(*iii*) **Pumped Storage Plants.** A pumped storage plant generates power during peak hours, but during the off-peak hours, water is pumped back from the tail water pool to the head water pool for future use. The pumps are run by some secondary power from some other plant in the system. The plant is thus primarily meant for assisting an existing thermal plant or some other hydel plant. A typical section for pumped storage plant is shown in Fig. 24.1.

During peak hours, the water flows from the reservoir to the turbine and electricity is generated. During off-peak hours, the excess power available from some other plant, is utilised for pumping water back from the tail pool to the head pool. This minor plant thus supplements the power of another major plant. In such a scheme, the same water is utilised again and again and no water is wasted.

For heads varying between 15 to 90 m, reversible pump turbines have been devised, which can function both as turbine as well as a pump. Such reversible turbines can work at relatively high efficiencies and can help in reducing the cost of such a plant. Similarly, the same electrical machine can be used both as a generator as well as a motor by reversing the poles. The provision of such a scheme helps considerably in improving the load factor of the power system.

(*iv*) **Tidal Plants.** Tidal plants for generation of electric power are the recent and modern advancements, and essentially work on the principle that there is a rise in sea water during high tide period and a fall *during the low ebb period.* The water rises and falls twice a day ; each fall cycle occupying about 12 hours and 25 minutes. The advantage of this rise and fall of water is taken in a tidal plant. In other words, the tidal range, *i.e.* the difference between high and low tide levels is utilised to generate power. This is accomplished by constructing a basin separated from the ocean by a partition wall and installing turbines in openings through this wall.

Water passes from the ocean to the basin during high tides, and thus running the turbines and generating electric power. During low tide, the water from the basin runs back to ocean, which can also be utilised to generate electric power, provided, special turbines which can generate power for either direction of flow are installed. Such plants are useful at places where tidal range is high. *Rance power station* in France is an example of this type of power station. The tidal range at this place is of the order of 11 metres. This power house contains 9 units of 38,000 k. watts.

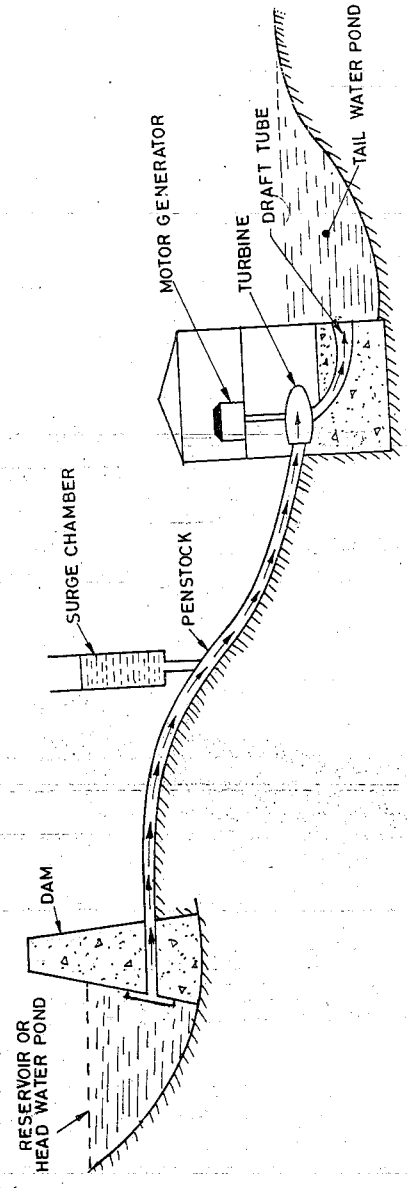


Fig. 24.1. Typical section *through a pumped storage plant.*

24.2.2. Classification of Hydroplants on the Basis of Operating Head on Turbines. On this basis, the plants may be divided into the following types :

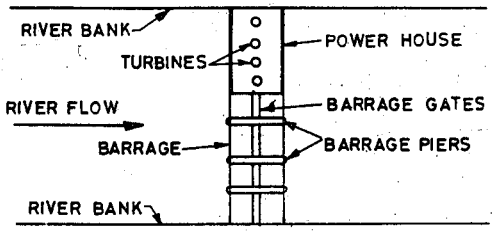
(*i*) *Low head scheme* (head < 15 m).

(*ii*) *Medium head scheme* (head varies between 15 to 60 m).

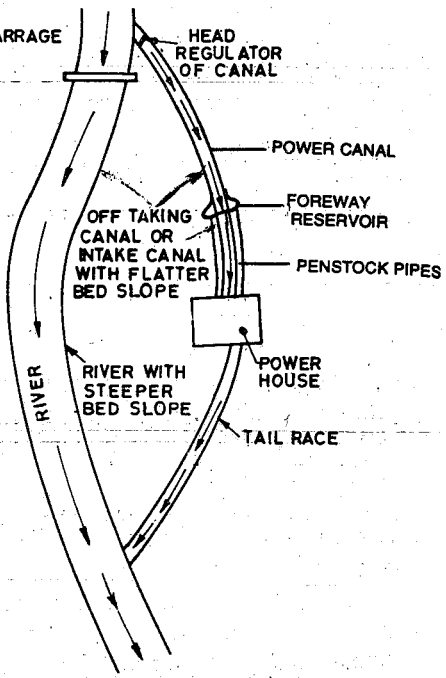
(*iii*) *High head scheme* (head > 60 m).

They are described below :

(*i*) **Low Head Scheme.** A low head scheme is one which uses water head of less than 15 metres or so. A run-off river plant is essentially a low head scheme. In this



(*a*) Run of river plant



(*b*) Diversion canal plant

Fig. 24.2. Low head schemes (in Plan)

scheme, a weir or a barrage is constructed to raise the water level, and the power house is constructed either in continuation with the barrage [Fig. 24.2 (*a*)] or at some distance downstream of the barrage, where water is taken to the power house through an intake canal [Fig. 24.3 (*b*)].

(*ii*) **Medium Head Scheme.** A medium head scheme is one which uses water head varying between 15 to 60 metres or so. This scheme is thus essentially a dam reservoir scheme, although the dam height is mediocre. This scheme is having features somewhere between low head scheme and high head scheme.

(*iii*) **High Head Scheme.** A high head scheme is the one which uses water head of more than 60 m or so. A dam of sufficient height is, therefore, required to be constructed, so as to store water on the upstream side and to utilise this water throughout the year. High head schemes up to heights of 1,800 metres have been developed. The common examples of such a scheme are : Bhakra dam in (Punjab), Rihand dam in (U.P.), Hoover

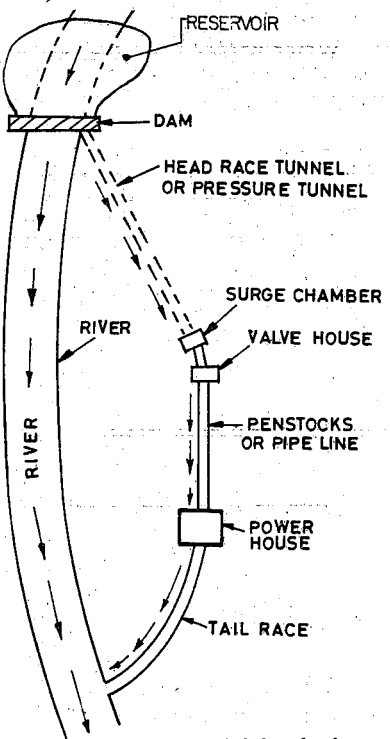


Fig. 24.3. (*a*) Plan of a high head scheme.

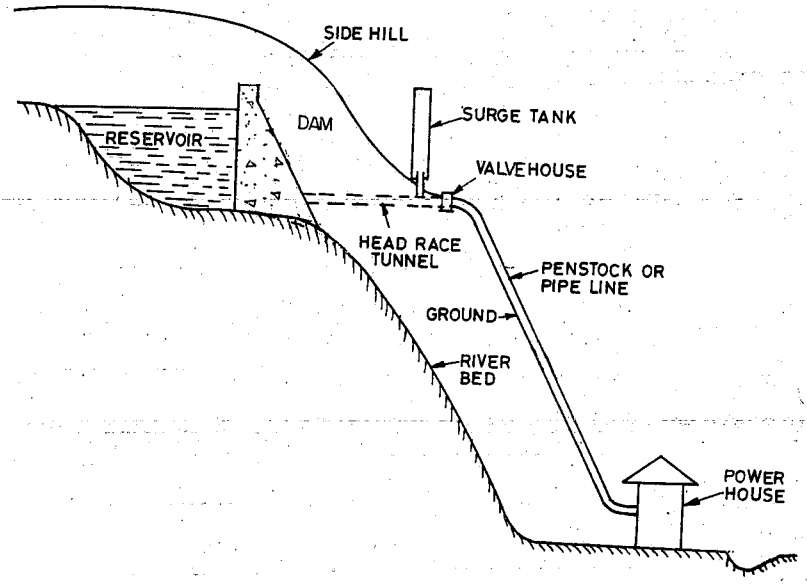


Fig. 24.3. (*b*) Section through a high head scheme.

dam in (U.S.A.), etc. A typical plan and section of such a scheme are shown in Figs. 24.3 (*a*) and (*b*).

The naturally available high falls can also be developed for generating electric power. The common examples of such power developments are : Jog Falls in India, and Niagara Falls in U.S.A.

24.3. Important Terms and Definitions Connected with Hydropower

24.3.1. Water Power Potential. The amount of power generated when Q cumecs of water is allowed to fall through a head difference of H metres is given by

$$\text{Water energy} = \gamma_w \cdot Q \cdot H \qquad ...(24.1)$$

where γ_w is unit wt of water $= 9.81$ kN/m^3

$= 9.81 \times Q \times H$ kN–m/sec. (*i.e.* k watts)

Electrical energy (E.E.) or Power in k. watts, is therefore, given as :

$$P = 9.81\,\eta \cdot Q \cdot H \text{ (k watts)} \qquad ...(24.2)$$

$\therefore$ Metric H.P. generated

$$= \frac{9.81\,\eta Q \times H}{0.735} = 13.33\,\eta QH \qquad (\because 1 \text{ m H.P.} = 0.735 \text{ kW}) \qquad ...(24.2a)$$

where η is the overall efficiency of turbine, generator, etc.

Using 80% efficiency, we get

Electrical energy $= 9.81 \times 0.8 \times Q \cdot H$ kW

$$= 7.84 \cdot Q \cdot H \approx \mathbf{8\ QH\ kW} \qquad ...(24.2b)$$

where, H is the design head in metres ; Q is the design discharge in cumecs.

24.3.2. Normal Water Level (N.W.L.). The highest elevation of water level that can be maintained in the reservoir without any spillway discharge, either with a gated or a non-gated spillway, is known as Normal water level.

24.3.3. Minimum Water Level (M.W.L.). The elevation of water level which produces minimum net head on the power units (*i.e.* 65% of design head H) is known as minimum water level.

24.3.4. Weighted Average Level (W.A.L.). The level above and below which equal amounts of power are developed during an average year (*i.e.* 50% units between N.W.L. and W.A.L., and 50% units between W.A.L. and M.W.L.) is called weighted average level.

24.3.5. Design Head. It is the net head under which the turbine reaches peak efficiency at synchronous speed. Generally, the design head = W.A.L. – M.W.L. The difference of N.W.L. and M.W.L. is 125% of design head.

24.3.6. Rated Head. It is the head at which the turbine functioning at full gate opening will produce a power output, equal to that specified in the name plate of the turbine. This rated head should be equal to the design head of the turbine, so as to ensure maximum overall plant efficiency.

24.3.7. Gross Head. Gross head is the difference in the water level elevations at the point of diversion of water for the hydel scheme and the point of return of water back to the river.

24.3.8. Operating Head. Operating head is the simultaneous difference between the elevations of water surface in the *foreway* and the *tailrace*, after making due allowance for approach and exit velocity heads.

$\therefore$ Operating head = T.E.L. at forway entrance – T.E.L. at tailrace exit. ...(24.3)

24.3.9. Net Head or Effective Head. The effective head is the net head applied to the turbine, and is given by the difference of head at the point of entry and exit of turbine, and includes the respective velocity and pressure heads at both places.

24.3.10. Installed Capacity of the Power House. The total capacity in kilowatts or million killowatts of all the turbine-generator units installed in a power house, is called its Installed capacity.

24.3.11. Dependable Capacity. It is the load carrying capability of the power house with respect to the load characteristics during a specified time interval, and is decided by the power factor, capability and the load on the power house.

24.3.12. Load Factor (L.F.). Load factor is defined as the ratio of the average load over a certain period of time to the peak load during the same period. Depending upon the period chosen, we may have different load factors, such as daily, monthly or annual load factors.

$$\therefore \quad \text{Load factor} = \frac{\text{Average load over a certain period}}{\text{Peak load during that period}} \qquad ...(24.4)$$

If the area under a load curve* is plotted, it would evidently represent the energy consumed in kilowatt hours (kWh). Thus, an annual load factor may also be defined as the ratio of the actual energy consumed during that year to the peak demand assumed to continue for one year.

* A graph plotted between time in hrs (on x-axis) and load in kW (on y-axis) is called a **load curve.**

∴ **Annual load factor**

$$= \frac{\text{Total yearly electrical units (kWh) produced}}{(\text{Max. power demand in kW}) \times 365 \times 24} \qquad ...(24.5)$$

Note. The maximum load determines the capacity of the units, while the load factor gives an idea of the degree of utilisation of the capacity. For example, an annual load factor of 0.6 would indicate that the machines are producing 60% of their yearly rated capacity (maximum production capacity).

24.3.13. Demand Factor. A consumer is provided with the connected load of a certain rating (kilo-watts). He may or may not use the whole load at one time, as the may use only a part of it. The ratio of the maximum demand at any particular time to the connected load is known as the *demand factor.*

24.3.14. Capacity Factor or Plant Factor. It may be defined as the ratio of average output of the plant for a given period of time to the plant capacity, *i.e.*,

$$\text{Capacity factor} = \frac{\text{Average Load(over a given period of time)}}{\text{Plant capacity}} \qquad ...(24.6\ a)$$

In other words, the capacity factor is the ratio of the energy actually produced by the plant in any given period to the energy it would be capable of producing at its full capacity during that period. For example, if a plant with a capacity of 10,000 kW were to produce 40,000 kWh when operating for 100 hours, then

$$\text{Capacity factor} = \frac{\frac{40,000}{100}}{10,000} = 0.4 \text{ or } 40\%$$

Hence, capacity factor

$$= \frac{\text{Energy actually produced in a given time}}{\text{Max. energy that can be produced by the plant during that given time}} \qquad ...(24.6\ b)$$

The capacity factor and load factor would become identical, if the peak load is equal to the plant capacity ; *i.e.* when plant is allowed to be used up to its max. capacity, or there is no *reserve capacity.* However, load factor would be different, if the plant is not used to full capacity.

Hence, in the above example, if the peak load is equal to 8000 kW (as against the station capacity of 10,000 kW), then

$$\text{Load factor} = \frac{\frac{40,000}{100}}{8,000} = 50\% \text{ as against the capacity factor of } 40\%.$$

Also, the **reserve capacity** would then be

$$\text{Peak load} = \frac{\text{Plant capacity}}{1 + R} \qquad ...(24.7)$$

where R is Reserve capacity expressed as fraction of Plant capacity,

$$\therefore \qquad 8000 = \frac{10,000}{1 + R}$$

or $$1 + R = \frac{10,000}{8,000}$$

or $$R = 0.25, \textit{i.e.}\ 25\%.$$

(*i.e.* at 25% R.C. ; Peak load × 1.25 = Installed capacity

24.3.15. Utilisation Factor or Plant Use Factor. It is defined as :

$$\text{Utilisation factor (U.F.)} = \frac{\text{Water actually utilised for power production}}{\text{Water available in the river}} \quad ...(24.8)$$

It the water head is assumed to be constant, then the utilisation factor would be equivalent to :

$$\text{U.F.} = \frac{\text{Max. power utilised}}{\text{Max. power available}} \quad ...(24.9)$$

The value of utilisation factor usually varies from 0.4 to 0.9 for a hydel plant, depending upon the plant capacity, load factor and storage.

24.3.16. Firm Power. The net amount of power which is continuously available from a plant without any break on firm or on guaranteed basis is known as *firm power.* This power should be available under the most adverse hydraulic conditions. The consumers can always be sure of getting this power.

24.3.17. Secondary Power. The excess power available over the firm power during the off peak hours or during monsoon, etc. is known as *secondary power.* There is no guarantee for secondary power, and it is supplied to the consumers on 'as and when available' basis.

24.3.18. Power Factor. It is defined as

$$\text{Power factor} = \frac{\text{Actual power in kilowatts (kW)}}{\text{Apparent power in kilo volt–amperes (KVA)}} \quad ...(24.10)$$

The power factor can never be greater than unity. Its value depends upon the relationship between the inductance and resistance in the load. A load with very little inductance such as lighting bulbs, will have a power factor close to unity. The usual system load has a power factor varying between 0.8 to 0.9, but if various induction motors are installed in the load, the power factor may be as low as 0.5. Since the electrical machines are generally rated in KVA, the actual power developed depends much upon the power factor. This is the reason, why most of the Electric Supply Undertakings, do stress upon their consumers, to reduce their inductance and improve their power factors.

Example 24.1. *The load on a hydel plant varies from a minimum of 10,000 kW to a maximum of 35,000 kW. Two turbo-generators of capacities 22,000 kW each have been installed. Calculate :*

(a) total installed capacity of the plant ;

(b) plant factor ;

(c) maximum demand ;

(d) load factor ;

(e) utilisation factor.

Solution. (*a*) Since two generators, each of capacity 22000 kW are installed, we have

the *total installed capacity* $= 2 \times 22000\ \text{kW} = 44000\ \text{kW}$. **Ans.**

From eqn. (24.6*b*), we have

(*b*) *Plant factor* (*i.e.* capacity factor)

$$= \frac{\text{Energy actually produced in time } t}{\text{Max. energy that can be produced in time } t}$$

$$= \frac{\frac{(10000+35000)}{2} \times t}{44000 \times t} = \frac{22500}{44000} = 0.511, \textit{i.e.}\ 51.1\%. \textbf{ Ans.}$$

(*c*) *Maximum demand* = 35000 kW (as given). **Ans.**

(*d*) *Load factor* $= \frac{\text{Average load over a certain period}}{\text{Peak load during that period}}$

$$= \frac{\frac{10000+35000}{2}\text{ kW}}{35000\text{ kW}} = \frac{22500}{35000} = 0.643 \textit{ i.e. } 64.3\%. \textbf{ Ans.}$$

(*e*) *Utilisation factor* $= \frac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{35000\text{ kW}}{44000\text{ kW}} = 0.795, \textit{i.e.}\ 79.5\%. \textbf{ Ans.}$$

Example 24.2. *A common load is shared by two hydel stations ; one being a base load station with 20 MW installed capacity, and the other being a stand-by station with 25 MW capacity. The yearly output of the stand-by station is* 10×10^6 *kWh and that of the base load plant as* 110×10^6 *kWh. The peak load taken by stand-by station is 12 MW and this station works for 2500 hours during the year. The base load station takes a peak of 18 MW. Find out :*

(a) Annual load factors for both stations.

(b) Plant use factors for both stations.

(c) Capacity factors for both stations.

Solution. We will work out all the required three factors for the base load plant first, and then for the other stand-by plant.

For Base Load Plant. For this base load station, data is given as :

Installed capacity $= 20\text{ MW} = 20 \times 10^3\text{ kW}$

Yearly output $= 110 \times 10^6\text{ kWh}$

Peak load taken $= 18\text{ MW} = 18 \times 10^3\text{ kW}$.

(*a*) Now, *Annual load factor*

$$= \frac{\text{Total energy (kWh) generated per year}}{\text{Max. power demand in kW} \times 365 \times 24}$$

$$= \frac{110 \times 10^6}{(18 \times 10^3) \times 365 \times 24} = \mathbf{0.698}, \textit{i.e.}\ \mathbf{69.8\%}. \textbf{ Ans.}$$

(*b*) *Plant use factor* $= \frac{\text{Max. power utilised}}{\text{Max. power available}}$

$$= \frac{18000\text{ kW}}{20 \times 10^3\text{ kW}} = \mathbf{0.9}, \textit{i.e.}\ \mathbf{90\%}. \textbf{ Ans.}$$

(*c*) *Capacity factor* $= \dfrac{\text{Av. energy utilised in a period (year)}}{\text{Max. energy that can be produced in that period}}$

$$= \frac{110 \times 10^6}{(20 \times 10^3) \times (24 \times 365)} = \mathbf{0.628}, \textit{i.e.}\ \mathbf{62.8\%} \quad \textbf{Ans.}$$

For stand by plant. The data given for this plant is :

Installed capacity $= 25$ MW $= 25 \times 10^3$ kW

Yearly output in 2500 hrs $= 10 \times 10^6$ kWh

Peak load $= 12$ MW $= 12 \times 10^3$ kW.

No. of working hours during the year = 2500 hr.

(*a*) *Annual load factor*

$$= \frac{\text{Total kWh generated per year}}{\text{Max. power demand in kW} \times (24 \times 365)}$$

$$= \frac{10 \times 10^6 \text{ kWh}}{(12 \times 10^3 \text{ kW}) \times (24 \times 365 \text{ h})} = \mathbf{0.095}, \textit{i.e.}\ \mathbf{9.5\%}. \quad \textbf{Ans.}$$

(*b*) *Plant use factor* $= \dfrac{\text{Max. power utilised}}{\text{Max. available power}}$

$$= \frac{12 \times 10^3 \text{ kW}}{25 \times 10^3 \text{ kW}} = \mathbf{0.48}, \textit{i.e.}, \mathbf{48\%}. \quad \textbf{Ans.}$$

(*c*) *Capacity factor* $= \dfrac{\text{Av. output}}{\text{Station capacity}}$

$$= \frac{10 \times 10^6 \text{ kWh}}{(25 \times 10^3 \text{ kW}) \times 2500 \text{ h}} = \mathbf{0.16}, \textit{i.e.}\ \mathbf{16\%}. \quad \textbf{Ans.}$$

24.4. Principal Components of a Hydro-electric Scheme

A hydroelectric development scheme ordinarily includes a diversion structure, a conduit (penstock) to carry water to the turbines, turbines and governing mechanisms, generators, control and switching apparatus, housing for the equipment, transformers and transmission lines to the distribution centres. In addition to these major components, trash racks at the entrance to penstock, canal and penstock gates, a foreway, a surge tank, and other appurtenances may be required. A tailrace channel from the power house back to the river must be provided, if the power house is situated at such a place that the draft tubes cannot discharge water directly into the river.

No two power development schemes are exactly alike and each will have its own unique problems of design and construction. The choice of a particular type of a plant at a given site depends upon various factors, such as :

(*i*) general available topography of the area ;

(*ii*) available head ;

(*iii*) available flow ;

(*iv*) availability of other type of power stations in the vicinity :

(*v*) requirements of power for industries, etc.

(*vi*) Political influences of the area, etc.

The major components of a hydroelectric scheme are described below :

(1) **The Foreway.** A foreway is a storage basin or any other large body of water situated just infront of intake. Its main function is to temporarily store the water which is rejected by the plant due to reduced load during off-peak hours, and also to meet the instantaneous increased demand when the load is suddenly increased. The foreway, therefore, absorbs the short interval variations in water demand and corresponding fluctuations in power load.

In many cases, when canal leads the water to the turbines, the canal itself may be large enough to absorb these flow variations. When the canal is long, its section is sometimes enlarged at the end, so as to provide necessary temporary storage. In many other cases, when a reservoir is constructed by building a dam, and the water is taken to the power house directly from the reservoir through penstocks, the reservoir itself acts as a foreway.

(2) **Intake Structure.** The foreway is provided with some type of intake structure, to direct water into the penstocks. Various types of intakes are used depending upon the local conditions.

Intakes should be provided with **trash racks** so as to prevent the entry of debris into the penstocks, and thus to avoid the possible damage to *wicket gates* and turbine runners or to avoid the choking up of the nozzles of the impulse turbines. They are made of thin flat steel bars placed 10 to 30 cm apart. The permissible velocity of water entering the trash racks is 0.6 to 1.6 m/sec.

A floating boom is often placed in the storage reservoir so as to trap as much ice and floating debris as possible, and thus to avoid their entry into the canal. At places, where severe winters do occur, provision is made so as to minimise the ice trouble in the foreway. The trash racks may be electrically heated, so as to prevent clinging of ice with them. Sometimes, an air-bubbler system which agitates the water in the vicinity of the trash rack and brings warmer water to the surface, is also used. The problem of ice automatically gets reduced, if the foreway pond is large enough, and the intake is at considerable depth.

The foreway must be provided with a spillway or a wasteway, a bypass channel, etc., so that the water can be disposed of safely, if such a need arises at any time. Syphon spillways may often be used advantageously for this purpose.

Besides trash racks, other accessories of an intake structure are :

(*i*) *Rakes and trolley arrangements* required to clear trash racks.

(*ii*) *Ice removal equipment,* such as a *floating boom, electrical heating device* for trash racks, etc., if necessary.

(*iii*) *Penstock closing gates* with their hoisting mechanisms.

(3) **Penstocks.** Penstocks are the huge diameter pipes which carry water under pressure from the storage reservoir to the turbines. The structural design of a penstock is essentially similar to that of any other pressure pipe. But, because of the possibility of sudden load changes, design against water hammer* is essential. Short length penstocks are generally designed to take this extra pressure by providing heavier pipe

* Explained in previous chapter.

wall. Slow-closing valves are then provided, so as to reduce these extra pressures. However, in case of long penstocks, a *surge tank* is generally provided, so as to absorb water hammer pressures and to provide water to meet sudden load increases.

Penstocks are generally made of steel or reinforced concrete, though wooden-stave pipes are also sometimes used. If the distance of the power house from the foreway is small, separate penstocks are generally used for each turbine. But long penstocks are generally branched at the lower end, so as to serve several turbines.

Penstock closing gates are usually provided at the entrance in the foreway. The gate can be closed to permit repair of penstock. An air vent and vent pipe connecting the top of the penstock with the open air, is provided downstream from the gate. Such a vent permits the entry of air into the penstock, as soon as the gates are closed and water drawn off through the turbine wheels. In the absence of such an arrangement, serious negative pressures will be exerted on the penstocks and may cause the penstocks to collapse. The water in the vent pipe should not get frozen, otherwise, entry of air will be prevented. Before opening the main head gate, the penstock, if empty, should be filled through a bypass, so that surge of water may not damage the turbines.

The penstocks should be located at such a level that sufficient water depth (called **water seal**) is provided above the penstock entrance in the foreway. If this is not so, and too little water depth is available, vortices and whirlpools will tend to form, which may carry air into the penstocks and the turbine wheels, and thus lowering the turbine efficiency and causing undesirable pressure surges. The entrance of the penstocks should also be flared, so as to avoid any loss of head by contraction.

Sharp bends must be avoided in penstocks, because they cause loss of head and require special anchorages. The penstocks may either be buried under the ground or kept exposed. Exposed or uncovered penstocks are usually supported on cradles and carried across ravines on trestles. Uncovered penstocks are generally costlier because they require other appurtenances such as expansion joints, saddles, anchors, etc. ; but they are easily accessible for inspection and repairs. It is not advisable to partially bury the penstock pipes, as the region near the ground surface will be subjected to excessive deterioration due to local movements. In a few cases, the penstocks can be replaced by constructing a tunnel through a hill, if a considerable saving in length can be gained.

(4) **Surge Tank or Surge Chamber.** The simplest type of a surge chamber consists of a cylindrical chamber open to the atmosphere and connected to the penstocks, as close to the power house as possible (Fig. 24.4).

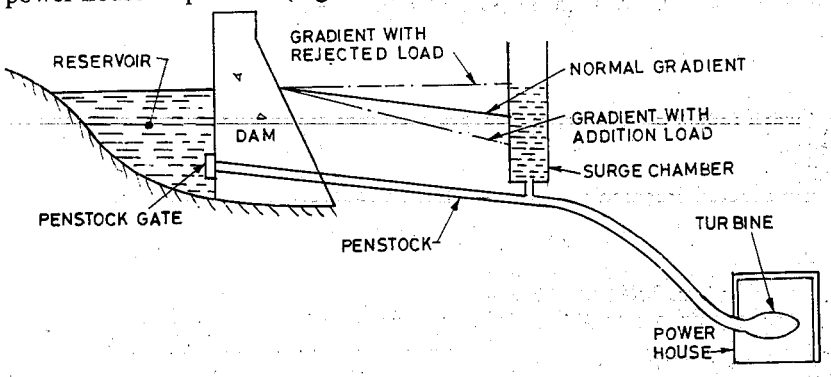


Fig. 24.4. Provisions of Surge chamber.

When the load is rejected by the power house turbines, the water level in the surge chamber rises, and decelerates the flow upstream of it. But when additional load comes, the immediate demand is met by drawing water from the surge chamber, which increases the flow gradient and thus accelerates the flow from the reservoir. When the load is steady, the water level in the surge chamber remains constant and at the level of normal hydraulic gradient line at that point, as shown in Fig. 24.4 A surge chamber, therefore, reduces the pressure fluctuations in the conduit pipe (*i.e.* penstocks) considerably, and thus prevents additional water hammer pressures from being exerted upon the walls of the conduit. The surge chamber should be high enough, to prevent any overflow of water, even at full load rejection.

Various types of surge chambers, such as (*i*) *Simple surge tank* (*ii*) *Throttled or Restricted orifice surge chamber,* (*iii*) *Differential surge chamber,* (*iv*) *Multiple surge chamber,* etc. are in use these days. The differential type of surge chamber is having a central riser pipe in addition to the main outer tank, as shown in Fig. 24.5.

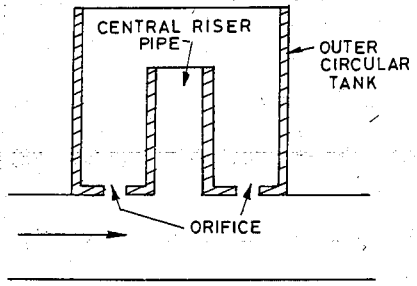


Fig. 24.5.
Differential type of surge chamber.

The main advantage of differential type of surge tank over a simple tank lies in the fact that retarding and accelerating heads are developed more promptly in the former case than in the latter case. Hence, for a given amount of stabilising effect, the capacity of a differential type of surge chamber may be less than that of a simple elementary type.

(5) **Hydraulic Turbines.** Turbines are machines which convert hydraulic energy into mechanical energy. The mechanical energy so developed by the turbine is then used to generate electric energy by directly coupling the shaft of the turbine with the generator.

In general, a turbine consists of a wheel (called runner) which is provided with specially designed blades or buckets. The water having large hydraulic energy is made to strike the runner, and thus causing it to rotate. This rotation of the turbine runner is passed on to the generator by coupling the generator and turbine together through the turbine shaft. This results in rotating the generator armature, and thus producing electrical power, called hydroelectric power.

Hydraulic turbines may be divided into two classes :

(*i*) Impulse turbines or Velocity turbines ; and

(*ii*) Reaction turbines or Pressure turbines.

These are discussed below :

(*i*) **Impulse turbines.** The important example of an impulse type of turbine is a *Pelton's wheel* or *a Pelton's turbine*. In such a turbine, all the available potential energy of water is converted into kinetic energy by passing the penstock water

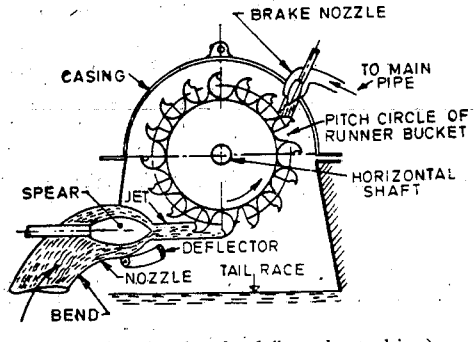


Fig. 24.6. Pelton's wheel (impulse turbine).

through a single nozzle. The water coming out of the nozzle in the form of free jet is made to strike on a series of buckets mounted on the periphery of a wheel (Fig. 24.6). This causes the wheel to revolve in open air, and water is in contact with only a part of the wheel at a time. The pressure of water is all along atmospheric, and thus there is no difference of pressure in the water at the inlet to the runner and at the outlet discharge to the tailrace. It also acts as a safeguard against accidents.

An impulse turbine is essentially a low speed turbine and is used for high heads of the order of 150 to 1,000 m (though heads of 1,700 to 1,800 m have been used in the world). Since it works under high heads, comparatively less quantity of water is required. It is, therefore used for high heads and low discharges.

(*ii*) **Reaction turbines.** The important examples of reaction turbines are : (*i*) *Francis turbine* ; and (*ii*) *Kaplan turbine.* A reaction turbine is one in which only a part of the potential energy of water is converted into velocity head (*i.e.* kinetic energy) and the balance remains as pressure head. Thus, the water entering the turbine possesses pressure as well as kinetic energy. The wheel is rotated under the action of both these forces. The water leaving the turbine also contains some pressure as well as velocity head. The pressure at the inlet is much higher than the pressure at the outlet. Since the entire flow takes place under pressure, a closed casing is absolutely necessary, so as to prevent any access of atmospheric air into the turbine. Since the water flows under pressure through such a turbine, the wheels of this turbine are submerged, and water enters all around the periphery of the wheel.

Difference between Pelton's (*i.e.* Impulse type) and **Francis** (*i.e.* Reaction type) **Turbines**

(1) In a Pelton's wheel, the total potential head is changed into kinetic head for affecting the motion of the runner ; while in a Francis wheel, only a part of it is converted.

(2) Water strikes only a few buckets at a time in Pelton's wheel ; while in Francis wheel, the water flows like that in a closed conduit. The runner is always full of water, and thus all the blades are simultaneously striken by water.

(3) In Pelton's wheel, the water falls freely to the atmosphere ; while in Francis wheel, the water is taken upto the tailrace by means of a closed draft tube, and thus, the whole passage of water is totally enclosed.

Classification of turbines according to specific speed

The specific speed (N_s) of a turbine is the speed of geometrically similar turbine that would develop one metric horse power under a head of one metre. Specific speed is given by the equation

$$N_s = \frac{N\sqrt{P_t}}{H^{5/4}} \qquad ...(24.1)$$

where N = Normal working speed of the turbine in RPM

P_t = The turbine output in metric horse power

H = Net or effective head in metres.

Table 24.1 shows the specific speeds of various turbines.

Table 24.1. Specific Speeds of Various Turbines

S. No.	*Type of turbine*	*Limit of head in metres*	*Specific Speed in Revolution per minute [i.e. R.P.M.]*
1.	Pelton turbine	Over 150 (High heads)	10 to 35
2.	Francis turbine	25 to 450 (Medium heads)	60 to 300
3.	Kaplan and Propeller turbines*	1 to 70 (Low heads)	300 to 1,000

Choice of a particular type of turbine

The selection of the type of a turbine primarily depends on head and load conditions. Different types of turbines can work satisfactorily only within a certain range of specific speeds (Table 24.1). At specific speeds greater than this usual range, there is a possibility of cavitation on the runner blades.

The variability of load also dictates the choice of a particular type of turbine. Say for example, for a head of 150 to 450 m or so ; from table 24.1, both Pelton's as well as Francis turbines can be used. But here the variability of load will influence the choice of a particular type. For higher range of heads, Pelton's wheel is preferred for part load operation in comparison to Francis turbine, although the Pelton's turbine involves higher initial cost. The quality of water also sometimes dictates the type of turbine to be chosen. For example, in the head range of 150 to 450 m, the choice of a Francis turbine is automatically ruled out, if sand is present in the water, because its runner will not be able to withstand its erosive action.

(6) **The Power House.** A power house is a building consisting of a *substructure* to support the hydraulic and electrical equipment and a *superstructure* to house and protect this equipment. For most of the plants which are equipped with reaction turbines, the substructure usually consists of a concrete block extending from the foundation to the generator floor with waterways (*i.e.* the scroll casings and draft tubes) formed within it. They are cast integrally while pouring concrete. The elevation of the turbine with respect to tail water level should be determined by the necessity of avoiding cavitation.

The superstructure is a building which generally accommodates the generators and excitors on the ground floor, and the switch board and control room on the mezzanine floor. Vertical turbines are placed just below the floor level beneath the generators, while horizontal turbines are placed on the ground floor along side the generators. The generating units are almost always arranged in a straight line across the direction of flow. A travelling crane spanning the width of the power house, is generally provided in every power house, so as to facilitate the lifting of heavy machines.

(7) **The Draft Tube.** The draft tube is a conduit which connects the outlet of a reaction turbine runner to the tailrace. Water, as it emerges out of the runner, flows through this pipe of gradually increasing diameter and comes to the tailrace level. The advantages of having a draft tube instead of letting the water fall directly into the atmosphere are :

(*i*) Effective pressure head is increased by an amount

$$= H_s + \frac{V_2^2 - V_3^2}{2g}$$

as explained below :

* Both Kaplan and Propeller turbines look alike and behave hydraulically in a similar fashion except that the Kaplan turbines have an additional arrangement of adjusting the inclination of the runner blades or vanes on the hub. This causes the Kaplan turbines to give higher efficiency even under part load conditions. *Kaplon turbines, cmpared to Propeller turbines, are therefore found to be more versatile, although costlier in their initial cost.*

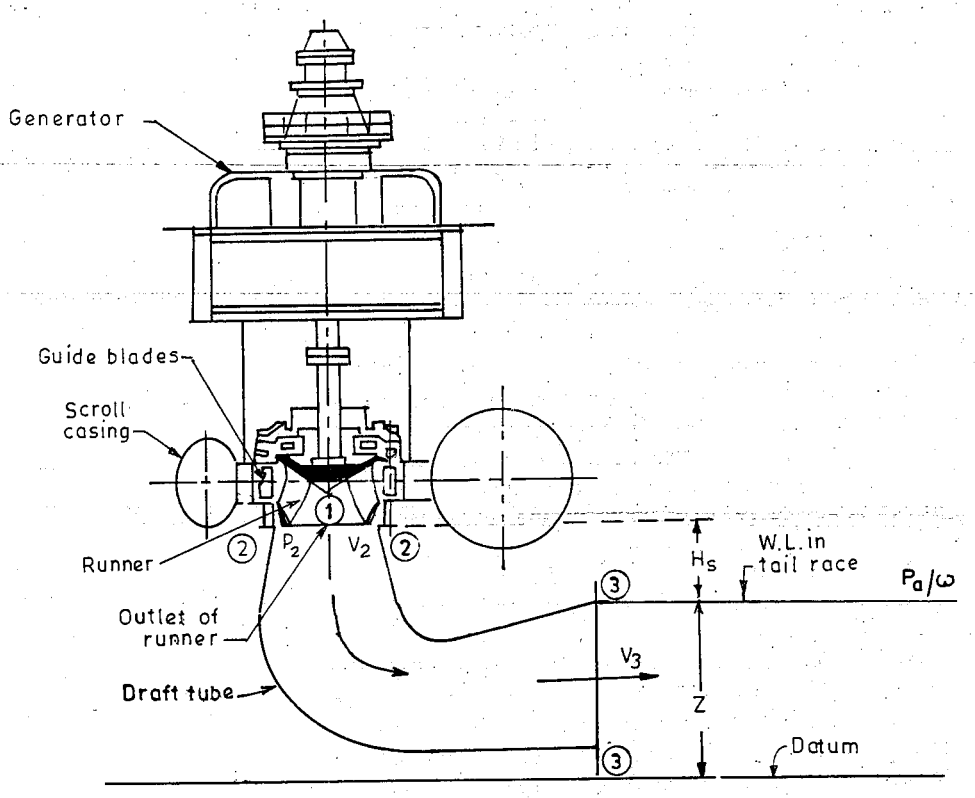


Fig. 24.7

Considering Fig. 24.7, we have

$$\frac{p_2}{\gamma_w}+Z+H_s+\frac{V_2^2}{2g}=\frac{p_a}{\gamma_w}+Z+\frac{V_2^2}{2g}$$

or

$$\frac{p_2}{\gamma_w}=\frac{p_a}{\gamma_w}+\frac{V_3^2}{2g}-\frac{V_2^2}{2g}-H_s=\frac{P_a}{\gamma_w}-\left(H_s+\frac{V_2^2}{2g}-\frac{V_3^2}{2g}\right)$$

Hence, the negative pressure created at the outlet of the runner

$$=H_s+\frac{V_2^2-V_3^2}{2g};$$

this should not exceed the vacuum pressure of water.

So if by gradually increasing the diameter of this pipe, we are able to reduce the velocity $V_3=0$, then the net increase in head

$$=H_s+\frac{V_2^2}{2g}.$$

(*ii*) The draft tube permits the turbines to be installed at a higher level than the tailwater level, which facilitates the maintenance, etc. of the turbine.

Outlet Gates. The outlet of the draft tube should be provided with gates, so that the draft tube can be dewatered for repairs. If the span between tailrace piers is not too high, timber logs may be used for this purpose. However, for larger spans, steel stop logs or

steel gates are commonly used. A hoisting mechanism is also required for lifting the gates or stop logs.

(8) **The Tailrace.** The channel into which the water is discharged after passing through the turbines is known as the *Tailrace*. If the power house is close to the stream, the outflow may be discharged directly into the stream. But when the stream is far off from the power house, one may have to construct a channel of considerable length between the power house and the stream. The tailrace must be designed properly and should not be neglected. In many low-head plants, more of the gross head may be utilised at very low cost if full attention is paid to the tailrace design.

Proper design and maintenance of the tailrace is necessary, so as to avoid excessive aggradation (*i.e.* silting) or degradation (*i.e.* scouring) of its bed. Aggradation will raise the tail water level and reduce the gross available head at the plant. The degradation will lower the tail water level, and the process may proceed up to a point where the water level falls below the top of the draft tube. This will lead to unsatisfactory flow conditions, increased losses, reduced turbine efficiency, and possibility of cavitation and consequent damage to the turbine blades.

Example 24.3. *A run-off river plant is to be constructed across a river at a site where a net head of 22 m is available on the turbines. The river carries a sustained minimum flow of 26 cumecs as dry weather flow. Behind the power station, sufficient water pondage has been provided to supply daily peak load of demand with a load factor of 70%. Assuming the plant efficiency of 58%, determine :*

(i) The maximum generating capacity of the generators to be installed at the power house.

(ii) The volume of pondage to be provided to supply the daily demand, assuming that the daily load pattern consists of average load for 21 hours and of peak load for 3 hours.

Solution. The power produced (at dry weather flow) is given by eqn. (24.2), as

$$P = 9.81\,\eta \cdot Q \cdot H \text{ (in kW)}$$

Here $\eta = 0.58$

$Q = 26$ cumecs

$H = 22$ m

$$\therefore \quad P = 9.81 \times 0.58 \times 26 \times 22 \text{ kW} = 3251 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Av. load (Av. power)}}{\text{Peak load (Peak power)}}$$

$$\therefore \quad 0.70 = \frac{3251}{\text{Peak load}}$$

$$\therefore \quad \text{Peak load} = \frac{3251}{0.7} = 4645 \text{ kW.}$$

(*i*) Assuming there is no reserve capacity, we have the *Maximum capacity of the generators to be installed* = 4645 kW. **Ans.**

(*ii*) Excess water from pondage is drawn in order to meet the excess demand (demand in excess of average) for 3 hours.

Excess power reqd. to be developed during 3 hours

$$= \text{Peak demand} - \text{Av. demand}$$

$$= 4645 - 3251 = 1394 \text{ kW}$$

Excess discharge required for developing this excess power is given as

$$P = 9.81\,\eta \cdot Q \cdot H$$

$$\therefore \quad 1394 = 9.81 \times 0.58 \times Q \times 22$$

$$\therefore \quad Q = \frac{1394}{9.81 \times 0.58 \times 22} = 11.15 \text{ m}^3/\text{sec.}$$

This excess discharge is required for 3 hours each day.

$\therefore$ Required pondays per day

$$= 11.15 \times 3 \times 60 \times 60 = 12.04 \times 10^4 \text{ m}^3 \quad \textbf{Ans.}$$

Example 24.4. *A run-off river plant with an installed capacity of 15,000 kW operates at 28% load factor when it serves as a peak load station :*

(a) What should be the minimum discharge in the stream, so that it may serve as a base load station ? The plant efficiency may be assumed to be 80% when working under a head of 20 m.

(b) Also calculate the maximum load factor of the plant when the discharge in the stream is 35 cumecs.

Solution. Installed capacity = 15,000 kW

$$\text{Load factor} = 28\%$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}}$$

The installed capacity plant serving as peak load plant means that the Installed capacity = Peak load.

$$\therefore \quad \text{Peak load} = \text{Installed capacity} = 15000 \text{ kW}$$

$$\therefore \quad 0.28 = \frac{\text{Av. load}}{15000}$$

$$\therefore \quad \text{Av. load} = 15000 \times 0.28 = \textbf{4200 kW.}$$

(*a*) Now, when this plant has to serve as base load plant, it should develop average load, *i.e.* 4200 kW.

Hence, Q reqd. to develop 4200 kW is given by :

$$P = 9.81\,\eta \cdot Q \cdot H$$

or $$4200 = 9.81 \times 0.8 \times Q \times 20$$

$$\therefore \quad Q = \frac{4200}{9.81 \times 0.8 \times 20} = \textbf{26.78 cumecs.} \quad \textbf{Ans.}$$

(*b*) When discharge in the stream is 35 cumecs, then power developed

$$= 9.81\,\eta \cdot QH$$

$$= 9.81 \times 0.8 \times 35 \times 20 \text{ kW} = \textbf{5488 kW.}$$

$$\text{Load factor} = \frac{\text{Av. power}}{\text{Peak power}} = \frac{5488 \text{ kW}}{15000 \text{ kW}} = 0.366, \textit{ i.e. } 36.6\%.$$

Hence, maximum load factor at 35 cumecs flow in the stream = **36.6 %.** **Ans.**

Example 24.5. *A run-off river plant is installed on a river having a minimum flow of 15 m^3/sec. If the plant is used as a peak load plant operating only for 6 hours daily,* compute the firm capacity of the plant :*

(*a*) *without pondage ;*

* Also called, *6 hours peaking plant.*

(b) with pondage but allowing 8% water to be lost in evaporation and other losses. Head at the plant is 16 m, and the plant efficiency may be assumed as 80%.

Solution. (*a*) Power developed by 15 m^3/sec discharge is given by

$$P = 9.81\,\eta \cdot Q \cdot H$$
$$Q = 9.81 \times 0.8 \times 15 \times 16 = \mathbf{1882\ kW.}$$

This represents the firm capacity of the plant without pondage. **Ans.**

(*b*) With pondage, the total volume of water stored during 18 hours when the plant is not operating is

$$= 15 \times (18 \times 60 \times 60)\ m^3 = 9.72 \times 10^5\ m^3$$

The loss of water due to evaporation, etc., per day

$$= 10\% \times 9.72 \times 10^5\ m^3 = 0.972 \times 10^5\ m^3$$

Net amount of available pondage per day

$$= (9.72 - 0.972)\ 10^5\ m^3 = 8.75 \times 10^5\ m^3$$

This daily pondage can supply a uniform Q, (reqd. for 6 hrs. only)

$$= \frac{8.75 \times 10^5}{6 \times 60 \times 60}\ m^3/sec = 40.51\ m^3/sec.$$

$\therefore$ Total flow available for power generation (reqd. only for 6 hrs)

$$= (15 + 40.51)\ m^3/sec = 55.51\ m^3/sec$$

$\therefore$ Power developed due to this discharge (*i.e.* firm power, because discharge is firm)

$$= 9.81 \cdot \eta \cdot Q \cdot H = 9.81 \times 0.8 \times 55.51 \times 16 = 6963\ kW.$$

Hence, the firm power developed by the plant (if pondage during rest hours is allowed) = **6963 kW.** **Ans.**

Example 24.6. *During a low water week, a river stream gets an average daily flow of 35 m^3/sec with daily fluctuations requiring a pondage capacity of about 16% of the daily flow.*

A hydroelectric plant is to be located on this river, which will operate for 5 days a week, 24 hours a day, but will supply power at varying rates, such that the daily load factor is of 60%, corresponding to which the pondage required is 0.2 times the meanflow to the turbines. On saturdays and sundays, all the flow is ponded for use on rest of the days.

If the effective head on the turbine, when the pond is full is to be 26 m, and the maximum allowable fluctuation in pond level is 1.2 m, determine :

(i) the surface area of the pond to satisfy all the operating conditions ;

(ii) the weekly output at the switchboards in kWh.

Assume the turbine efficiency as 82% and generator efficiency as 90%.

Solution. $Q = 35\ m^3/sec.$

Now, Daily flow $= 35 \times 24 \times 60 \times 60 = 3.024 \times 10^6\ m^3$

Pondage reqd. for discharge fluctuations

$$= 16\% \text{ of daily flow} = \frac{16}{100} \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{0.484 \times 10^6 \text{ m}^3} \quad ...(i)$$

The discharge at the turbines is caused by the daily flow of 35 cumecs plus the discharge caused by the water stored during the two weekly rest days, *i.e.* saturday and sunday.

Now, *total water stored in 2 rest days*

$$= 2 \times \text{Daily flow} = 2 \times 3.024 \times 10^6 \text{ m}^3$$

$$= \mathbf{6.048 \times 10^6 \text{ m}^3} \quad ...(ii)$$

This stored water is utilised during the 5 working days in the week. Hence, the discharge obtained per day from this storage

$$= \frac{6.048 \times 10^6}{5} \text{ m}^3 = 1.2096 \times 10^6 \text{ m}^3.$$

The streamflow caused by this storage

$$= \frac{1.2096 \times 10^6}{24 \times 60 \times 60} \text{ m}^3/\text{sec} = 14 \text{ m}^3/\text{sec}.$$

$\therefore$ Total average flow to the turbines

$$= 35 + 14 = \mathbf{49 \text{ m}^3/\text{sec}}.$$

Hence, pondage required for load fluctuations

$$= 0.2 \times \text{Mean daily flow to turbines}$$

$$= 0.2 \times 49 \times (24 \times 60 \times 60) = \mathbf{0.847 \times 10^6 \text{ m}^3} \quad ...(iii)$$

Hence, the total ponding required

$$= (i) + (ii) + (iii)$$

$$= [0.484 \times 10^6 + 6.048 \times 10^6 + 0.847 \times 10^6] \text{ m}^3 = 7.379 \times 10^6 \text{ m}^3.$$

(*i*) With maximum variation of 1.2 m in pond level, we have the min. surface area of pond reqd. to accommodate 7.379×10^6 m^3 of water

$$= \frac{7.379 \times 10^6}{1.2} \text{ m}^2 = 6.15 \times 10^6 \text{ m}^2$$

$$= \frac{6.15 \times 10^6}{10^4} \text{ hectares} = \mathbf{615 \text{ hectares.}} \quad \textbf{Ans.}$$

Since the allowable max. fluctuation in pond level is 1.2 m, and the head at full level is 26 m, we have the average head on turbines

$$= \frac{26 + (26 - 1.2)}{2} = \frac{26 + 24.8}{2} = \frac{50.8}{2} = 25.4 \text{ m}.$$

(*ii*) $\therefore$ Max. Power generated $= 9.81 \eta \cdot Q \cdot H$.

where η is the combined efficiency of turbines and generators

$$= 0.82 \times 0.90 = 0.738$$

$$Q = 49 \text{ m}^3/\text{sec}$$

$$H = 25.4 \text{ m}$$

$\therefore$ $P_{max} = (9.81 \times 0.738 \times 49 \times 25.4)$ kW $= 9001.4$ kW.

With a daily load factor of 0.60, we have the average power developed = $0.6 \times$ Max. power

$$= 0.6 \times 9001.4 = 5400.8 \text{ kW.}$$

$\therefore$ Weekly power developed at the switchboard

$$= (5400.8 \times 5 \text{ days}) \text{ kW day}$$
$$= (5400.8 \times 5 \times 24) \text{ kWh} = 6,48,100 \text{ kWh}$$

Hence, weekly output at switchboard

$$= \mathbf{6.481 \times 10^5 \text{ kWh.}} \quad \textbf{Ans.}$$

Example 24.7. *A run-of-stream station with an installed capacity of 15000 kW operates at 15% load factor when it serves as a peak load station. What should be the lowest discharge in the stream so that the station may serve as the base load station ? It is given that the plant efficiency is 75% when working under a head of 20 m.*

(b) Also calculate the maximum load factor of the plant when the discharge in the stream rises to 20 cumecs.

Solution. Installed capacity = 15000 kW

Load factor = 15%

Since the plant acts as peak load station, the installed capacity would be equal to the peak load.

Now, $\text{Load factor} = \dfrac{\text{Average Load}}{\text{Peak Load}}$

$\therefore \quad 0.15 = \dfrac{\text{Average Load}}{15000}$

$\therefore$ Average load $= 15000 \times 0.15 = \mathbf{2250 \text{ kW.}}$

In order that the plant can act as a base load station, it must supply 2250 kW average power. Using

$P = 9.81 \eta \cdot Q \cdot H$, we have

$P = 2250$ kW

$\eta = 0.75$ (given)

$Q = ?$

$H = 20$ m.

$\therefore \quad 2250 = 9.81 \times 0.75 \times Q \times 20$

$\therefore \quad Q = \dfrac{2250}{9.81 \times 0.75 \times 20} = \mathbf{15.31. \text{ m}^3/\text{sec.}}$

Hence, the stream must carry a minimum discharge of 15.31 cumecs, in order to make the plant work as base load station. **Ans.**

(b) When $Q = 20$ cumecs, then power developed by the plant is given as :

$$P = 9.81 \eta \cdot QH$$
$$= 9.81 \times 0.75 \times 20 \times 20 = 2940 \text{ kW}$$

$$\text{Load factor} = \frac{\text{Av. Load}}{\text{Peak Load}} = \frac{2940 \text{ kW}}{15000 \text{ kW}} = 0.196, \textit{ i.e. } \mathbf{19.6\%.} \quad \textbf{Ans.}$$

Example 24.8. *The water turbines at a hydel storage plant produces 10,000 H.P.; when working under a net head of 30 m and with an overall efficiency of 80%. The inflow in the reservoir during a year is given below :*

Table 24.1

Month	*Inflow in million cu. m (M. cu. m)*
Jan.	90
Feb.	80
March	73
April	80
May	70
June	98
July	120
Aug.	80
Sep.	96
Oct.	105
Nov.	100
Dec.	75

Find (a) the minimum reservoir capacity reqd. to satisfy the uniform demand of water ; (b) the total quantity of water wasted during the year. Assume the reservoir to be full at the beginning of November. Use analytical method.

Solution. The H.P. produced by turbines = 10,000

Since 1 H.P. = 0.736 kW, we have the power produced by turbines in kW units

$$= 10{,}000 \times 0.736 = 7360 \text{ kW.}$$

Discharge reqd. to produce 7360 kW, Q, is given by

$$P = 9.81 \cdot \eta \cdot QH$$

where $\eta = 0.80$

$Q = ?$

$H = 30$ m

$$\therefore \quad 7360 = 9.81 \times 0.80 \times Q \times 30$$

or
$$Q = \frac{7360}{9.81 \times 0.80 \times 30} = 31.29 \text{ m}^3/\text{sec.}$$

$\therefore$ Flow reqd. during each month

$$= 31.29 \times (30 \times 24 \times 60 \times 60) = 81 \times 10^6 \text{ m}^3 = \mathbf{81 \text{ M. cum.}}$$

It shows that 81 M. cum of flow is required to be compulsarily released in order to produce the requisite electric power. The inflow and this reqd. uniform outflow data is now analysed in Table 24.2 starting from Nov. onward (when reservoir is full) to

Table 24.2

Month	*Inflow in M. cum*	*Outflow in M. cum*	*Draft used from storage in M. cum.*	*Storage refill in M. cum.*	*Reservior depletion in M. cum*	*Water wasted M. cum*
Nov.	100	81	0	—	—	19
Dec.	75	81	6	—	6	—
Jan.	90	81	—	6	0	3
Feb.	80	81	1	—	1	—
March	73	81	8	—	9	—
April	80	81	1	—	10	—
May	70	81	11	—	21	—
June	98	81	—	17	4	—
July	120	81	—	4	0	35
Aug.	80	81	1	—	1	—
Sept.	96	81	—	1	—	14
Oct.	105	81	—	—	—	24
Σ						95

workout the storage used, refill, net reservoir depletion, and the water wasted, as shown in this table. The table is otherwise self explanatory.

(*a*) From this table, it can be seen that the max. reservoir depletion is **21 M. cum**, which evidently represents the minimum reservoir capacity required to satisfy the uniform demand. **Ans.**

(*b*) Total quantity of water wasted = **95 M. cum. Ans.**

24.5. Comparison of Hydro-power with Thermal Power with Reference to Indian Conditions

Water is available in abundance in India, and most of our important cities and other industrial or populated areas are situated within a maximum distance of say 500 km. from the main concentrations of hydro-resources. Hydro-power can, therefore, be easily developed in India along with other utilisations of our water resources, such as irrigation, navigation, flood control, city water supply, etc. The hydro-power has ultimately been found to be cheaper than the thermal power, although its exploitation entails large initial expenditure. It has been estimated, that on an average, the initial investment on a hydro project is about 25% more than that on a thermal project. But, the recurring cost on a hydro project is much less than that on a thermal project, and hence, the hydropower finally comes out to be more economical. Besides these economic considerations, hydropower has been mainly produced in India as a side product during the development of river valleys for irrigation and flood control.

Hydropower is much more important than thermal power in India, mainly because we do have almost inexhaustible water resources in our country, and the same is available in the vicinity of most of our needy areas. In sharp contrast to this, the coal, which is the only cheap and conventional source of thermal power, is confined only to a few regions, and has to be transported over long distances. Hence, in areas which are far away from coal bearing regions, hydropower is the only choice. Moreover, the amount of coal available in our country, is limited in quantity and, therefore, cannot be used infinitely without proper planning for the future. It can be stated that due to various inherent advantages which the hydropower possesses over thermal power, the use of hydropower has been found justifiable on overall economic considerations even in areas close to the collieries in Andhra Pradesh, Madhya Pradesh, Orissa, etc. But in areas, where hydropower has also been fully tapped, we have to think of nuclear or other thermal resources. *Pumped storage plants* may also be thought of in such areas, and their possibilities should also be investigated.

In addition to these advantages, the hydro power is considered to be environmental friendly as compared to thermal power, since thermal power generation causes tremendous air pollution, directly affecting the health of the public.

Besides all these advantages of hydropower over thermal power, there is one serious limitation of hydropower. The limitation is that the hydro power production is totally dependent upon rainfall. Though hydro-projects are designed on a very conservative basis of planning on minimum water that would be available for 90 out of 100 years from a catchment, still there may be some periods when reservoirs do not fill up. If this happens throughout the country, the power position will become difficult. This has

happened in 1972 monsoon, when practically all the reservoirs were only half full, resulting in a sudden and drastic power shortage.

Taking such contingencies into account, it can finally be concluded that we must develop a combined system of power production. Hydropower should be exploited to as much extent as possible, and sufficient thermal power reserve should be kept to supplement the hydropower during the times of water crisis.

24.6. Hydropower Potentials of India

India has a vast potential for hydropower generation, particularly in the northern and north-eastern regions. As per an estimate made by the Central Electricity Authority, the potential of the country is assessed at 84 G.W.* at 60% load factor** (*i.e.* 140 GW of installed capacity), which is equivalent to about 440 billion units+ (kWh) of annual energy generation at a power factor of 0.6. The basin wise distribution of hydropower potential and developed hydropower is shown in Fig. 24.8.

The development of hydropower began in India, sometimes in the year 1897, when a 300 kW power station was installed at Darjeeling. It was followed by a power station at Sivasamundram in Mysore State, in 1902. It was then followed by a 50 MW Tata Hydro Electric project of Bombay City in 1914. The process of installation of hydro power projects was slow before independence, but it gathered a lot of momentum after India became free. So much so, that the hydropower production which was hardly 0.5 GW* before independence has gone to as high a value as 13.2 GW (at 60% load factor) by the end of 8th five year plan (*i.e.* March 1997). Another 6 GW shall be available from the projects which are under construction, and 3 GW from the projects which have been cleared for execution.

As stated above, the 1997 figure of developed power of 13.2 GW and the total potential of 84 GW are the values at 60% load factor**, which means that the presently installed capacity (1997) equals 13.2/0.6 = 22 GW, as against the potential (installedl capacity) of 84/0.6 = 140 GW, respectively, as shown in Fig. 24.8.

This estimated economically exploitable hydropower potential of 84 GW (at 60% L.F.) for India is quite low in comparison to the corresponding World figure of 4200 GW. Against our potential of 84 GW or 440 bkWh+, we have so for installed only 13.2 GW i.e. 69 bkWh++, which is only about 15.7% of the potential of 84 GW (440 bkWh). It evidently shows that a lot of work is still left to be done in this direction. The power potentials and power exploited on different river basins of India are shown in Fig. 24.8. It can be seen from this figure that Brahmaputra river basin has the maximum potential (35 GW), though least (0.5 GW) has been exploited. A lot of work is also left to be done on Indus and Ganga basins.

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* 1 GW (giga watt) = 10^9 watts = 1 million kW = 1 MkW
1 TW (trillion watt) = 10^{12} watts. = 10^9 kW = 1 billion kW = 1 b kW

** 60% load factor means that the installed machines are run, on an average, to give output capacity equal to 60% of the station capacity. This is so, because machines will not always be run at the installed station capacity but will be run, on an average, at 60% of this capacity.

\+ 84 GW is equivalent to = 84 × 365 × 24 (0.6 *i.e.* P.F) GWh = 441504 GWh : say **440 TWh. *i.e.* 440 b kWh.**

++ 13 GW is equivalent to 13 × 365 × 24 × 0.6 (P.F) GWh = 68328 GWh : say ≈ 68 b kWh

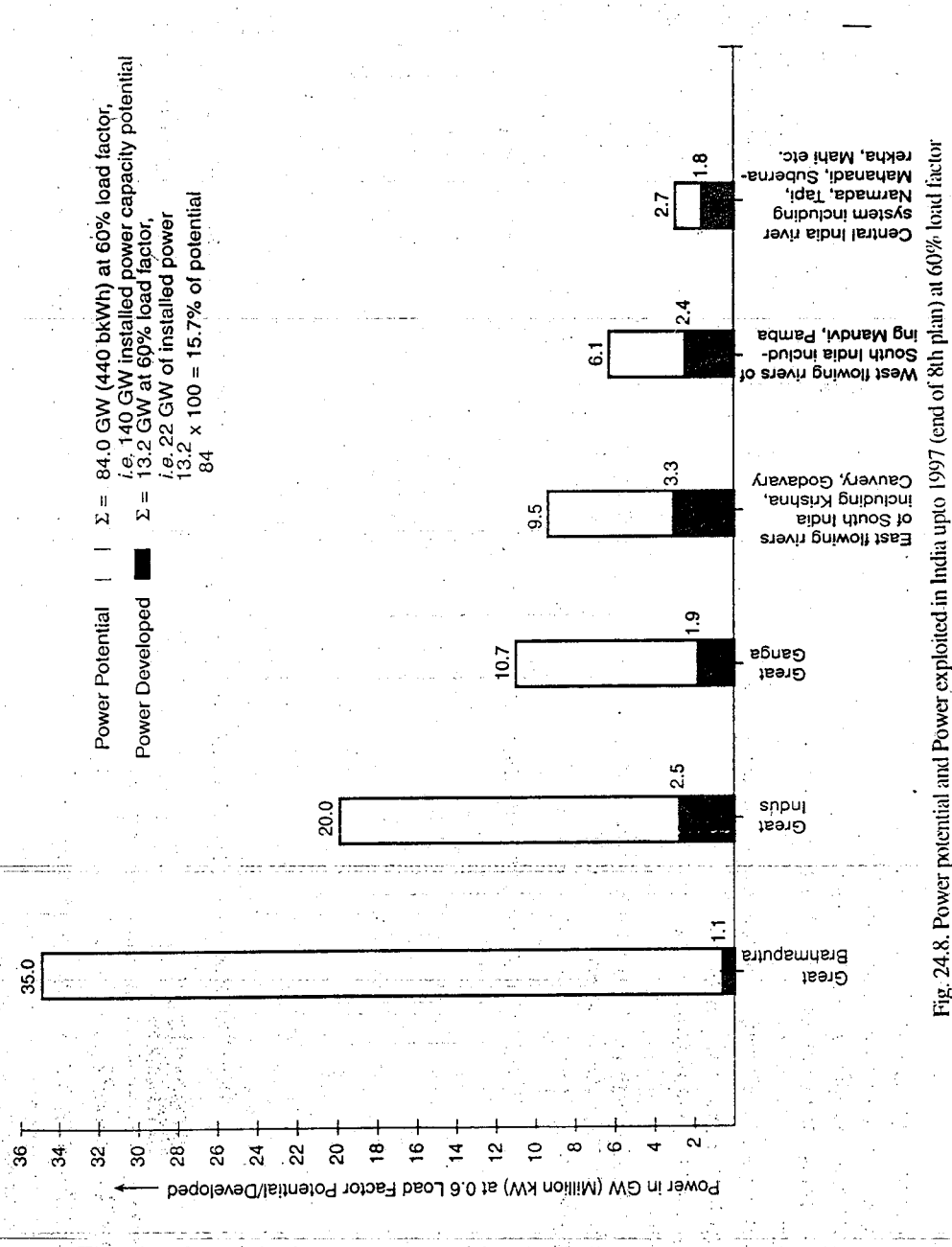


Fig. 24.8. Power potential and Power exploited in India upto 1997 (end of 8th plan) at 60% load factor

The developed hydropower capacities in India at the end of different plan periods in comparison to total power (*i.e.* hydro + thermal + nuclear) are indicated in table 24.3.

Table 24.3. Developed Hydro Power and Total Power in India at 60% Load Factor

Year end	*Installed capacity in GW (MkW)*		*%age of hydropower w.r. to total power*
	Hydropower	*Total power (Hydro + Thermal + Nuclear + Others)*	
1950	0.34	1.71	19.6
1956	0.64	2.89	22.02
1960-61	1.15	4.65	24.7
1965-66	2.47	9.03	27.4
1968-69	3.54	12.96	27.4
1974-75	4.52	18.32	24.7
1983-84	8.34	39.30	21.9
1985-86	9.30	46.80	19.9
1989-90	11.14	64.73	17.3
1996-97 (end of 8th plan)	13.2* (69 bkWh)	83.0** (435 bkWh)	15.9

*Against the potential target of 140 GW.
**Contains only about 2.5 MkW of nuclear power.

In India, the nuclear power generation is only about 3% of the total power generation, (including hydropower, thermal power, nuclear power and power from non-convention energy sources like wind power & solar power). The nuclear power is highly cost intensive*, Whereas thermal power involves huge fuel and *R* of *M* (Running of Maintenance) cost particularly for plants located far away from coal mines, or petro gas locations. The disposal of waste coal ash and air pollution caused by thermal power is also a costlier exercise, as compared to the present day cost being incurred for disposal of radioactive nuclear wastes, since such wastes can, through an expensive reprocessing, may lead to recovery of very costly plutonium, which may offset the cost of reprocessing and solve the problem of disposal of such nuclear radioactive wastes.

The Indian *Department of Atomic Energy* (DAE) is, therefore, planning to step up production of nuclear power as to boost its generation from 3% to 8% of the total electricity generation in the next 15 years. Whereas, the Deptt of Atomic Energy has calculated the nuclear power to be cheaper than the thermal power on an annual discounting of capital cost at about 8%, yet these calculations, based on various assumptions, have not been accepted by many, such as from Mr. Ramana, a fellow at *the centre for Interdisciplinary Studies in Environment and Development Bangalore,* and Mr. Amulya K.N. from the *International Energy Initiative*. Be that as it may, we need a mixed bag of various types of powers, since coal is an exhaustible source and cannot continue for ever. Hence, the aim of DAE to increase nuclear power production should not be opposed by saying that it is costlier than the thermal power or because of its high initial cost.

* The capital cost incurred on recent installation of two units of nuclear reactors of 220 MW each at the *Kaiga atomic power station (Kaiga I & II)* was about Rs. 2900 crores with additional initial fuel loading cost of Rs. 1045 crores for heavy water and Rs. 84 crores for Uranium, as compared to Rs. 612 crores spent on installing *Raichur Thermal Power Station VII*, both plants being of similar size, of recent vintage, and far away from coal supplies.

24.7. Electricity Generation from Various Sources in the World

The electricity generation from different sources in bkWh in various countries of the world are shown in table 24.4.

Table 24.4.Electricity Generation from Different Sources in Verious Countries of the World

Name of Country	*Total Electricity production in Billion kWh 1996*	*% age of production of electricity from different sources*				
		Hydro power % 1996	*Coal % 1996*	*Oil % 1996*	*Gas % 1996*	*Nuclear power % 1996*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
Algeria	20.7	0.7	—	3.6	**95.7**	—
Argentina	69.8	32.9	2.2	5.4	48.3	10.7
Australia	177.0	8.7	78.9	1.6	9.0	—
Austria	53.5	64.0	11.5	3.7	17.5	—
Bangladesh	11.5	6.4	—	6.5	87.1	—
Belarus	23.7	0.1	—	29.1	70.8	—
Belgium	75.2	0.3	24.2	1.7	14.6	57.6
Bolivia	3.2	63.0	—	6.2	29.6	—
Brazil	289.8	91.7	1.6	3.1	0.2	0.8
Bulgaria	41.5	4.0	41.9	3.2	7.2	43.6
Canada	570.6	62.4	16.2	1.6	2.9	16.3
Chile	30.8	54.8	35.1	8.4	1.0	—
China	1080.0	17.4	75.0	6.0	0.2	1.3
Hong Kong, China	28.4	—	**98.4**	1.6	—	—
Colombia	44.6	79.1	6.9	0.5	12.8	—
Cuba	13.2	0.7	—	92.2	0.1	—
Czech Republic	63.8	3.1	73.3	1.1	1.2	20.1
Denmark	53.6	0.0	74.0	10.8	10.7	—
Egypt, Arab Republic	57.6	18.8	—	37.1	44.1	—
Finland	69.4	17.1	31.8	1.9	12.3	28.1
France	508.1	12.8	6.1	1.5	0.8	**78.2**
Germany	550.6	4.0	55.0	1.4	8.7	29.1
Ghana	6.0	99.9	—	0.1	—	—
Greece	42.3	10.3	69.3	20.2	0.2	—
Guatemala	4.3	74.3	—	20.6	—	—
Hungary	35.1	0.6	28.3	12.6	18.1	40.4
India	**435.1**	**15.9**	**73.2**	**2.8**	**6.2**	**1.9**
Indonesia	67.1	13.3	25.9	25.4	32.0	—
Iran, Islamic Republic	90.9	8.1	—	37.1	54.7	—
Iraq	29.0	2.0	—	**98.0**	—	—
Ireland	18.9	3.8	48.4	14.2	33.2	—
Israel	32.5	0.2	68.9	30.9	—	—
Italy	239.5	17.6	10.6	48.9	21.0	—

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Jamaica	6.0	2.1	—	93.3	—	—
Japan	1003.2	8.0	18.2	21.0	20.2	30.1
Jordan	6.1	0.4	—	87.6	12.0	—
Kenya	4.0	81.6	—	8.8	—	—
Korea Dem Republic	35.0	64.3	35.7	—	—	—
Kuwait	25.5	—	—	21.7	78.3	—
Malaysia	51.4	10.1	6.4	12.3	71.2	—
Mauritius	—	—	—	—	—	—
Mexico	162.5	19.3	10.9	50.1	11.3	4.8
Morocco	12.4	15.7	44.9	39.4	—	—
Myanmar	4.3	38.3	0.1	15.9	45.7	—
Nepal	1.2	90.2	—	9.8	—	—
Netherlands	85.0	0.1	31.6	4.6	55.6	4.9
New Zealand	36.2	71.2	3.0	0.0	17.8	—
Niger	—	—	—	—	—	—
Nigeria	15.0	36.7	—	26.1	37.2	—
Norway	104.5	**99.2**	0.2	—	0.3	—
Pakistan	57.0	40.7	0.8	30.8	26.8	0.8
Peru	17.3	77.1	—	20.9	1.3	—
Philippines	36.7	19.3	13.2	49.6	0.1	—
Poland	141.2	1.4	96.8	1.2	0.3	—
Portugal	34.4	42.9	36.6	17.5	—	—
Romania	61.4	25.7	33.9	10.9	27.2	2.3
Russian Federation	846.3	18.2	18.5	9.4	40.4	12.9
Saudi Arabia	97.8	—	—	57.1	42.9	—
Singapore	24.1	—	—	78.7	18.6	—
South Africa	198.3	0.7	93.2	—	—	5.9
Spain	173.4	23.0	31.5	8.0	3.9	32.5
Sri Lanka	4.5	71.8	—	28.2	—	—
Sweden	139.6	36.9	3.0	5.2	0.3	52.5
Switzerland	55.6	51.0	0.0	0.5	1.2	45.2
Syrian Arab Republic	17.0	40.8	—	29.1	30.0	—
Thailand	87.5	8.4	20.0	39.3	42.0	—
Turkey	94.9	42.7	32.1	6.9	18.1	—
United Arab Emirates	19.7	—	—	17.1	82.9	—
United Kingdom	346.3	1.0	42.4	4.0	23.6	27.3
United States	**3652.0**	9.6	52.7	2.6	13.2	19.6
Vietnam	16.9	0.0	14.0	8.6	77.3	—

Note. Totals of Col. (2) to (7) infront of a particular country may not total to 100%, since other sources generating electricity, such as geothermal, solar & wind, are not shown here.

PROBLEMS

1. (*a*) What is meant by Hydro-power ? Compare hydro-power with thermal power w.r. to Indian conditions.

(*b*) Enumerate the different types of hydel plants, and describe the 'storage plant'.

2. (*a*) How do you classify a hydro-electric scheme on the basis of its operating head.

(*b*) Define and differentiate between the following in connection with hydropower :

(*i*) Firm and secondary power.

(*ii*) Load factor, utilisation factor and plant factor.

(*iii*) Installed and dependable capacity of a power house.

(*iv*) Design head, rated head, gross head, operating head and effective head.

3. What are the principal components of a hydro-electric scheme ? Discuss the utility of each component.

4. (*a*) Write a brief note on the use and types of 'turbines' in a hydroelectric scheme.

(*b*) What is a 'surge tank', and what are its types and uses ?

(*c*) What is a 'draft tube', and what are its uses ?

5. Write short notes on :

(*i*) Hydropower potential of India.

(*ii*) Hydropower *vs.* thermal power.

(*iii*) Surge tanks and their types.

(*iv*) Hydraulic turbines and their types.

(*v*) Trash racks.

25

River Navigation

25.1. Introduction

There shall be no exaggeration, if we define water as something which is a necessity, a comfort and a luxury too. There cannot be two opinions about water being a necessity, as there can be no life without water. Men, animals and even plants can never survive if there is no water. Besides the basic necessities for existence, water provides us various comforts and luxuries, and makes our lives easier. Besides the various uses of water, water provides us cheap* means of transport. Before the inventions of rail-road transport, water was the only means of communication between distant places. Even today, cargo – too heavy to be packed on wagon trains or trucks, is lifted and transported through big ships. In the modern affluent living, towing through water is not only required for transporting purposes, but is also required for recreational boating. However, boating and floating of ships through the natural rivers is not always safe. Rapids and sandbars may create problems, and may require considerable time to be passed. Isolated rocks, fallen trees, debris and other obstructions may create constant hazards, and may damage or even wreck the boats, steamers, or ships being towed through such waterways. It is, therefore, absolutely essential that all the waterways through which the boats or ships are to be towed, must be made completely safe.

The chief requirement for navigating through a waterway is the availability of sufficient water depth in the waterway. A minimum water depth of about 2.7 metres is generally required for navigating safely and economically ; although a depth of about 3.7 metres is generally aspired in the final development of a navigable waterway. Availability of lesser depth in the rivers may completely eliminate the possibility of towing the ships through such rivers or may cause increased unit cost of transport.

The cost of water transport is generally expressed in Rs. per tonne per kilometre. The total cost per kilometre of travelled distance divided by the transported tonneage, gives us the cost per tonne per kilometre. The total cost consists of fuel charges, wages of crews, etc., and other operating and incidental charges. The wages and other operating charges depend only upon the time of transit ; while the fuel consumption charges depend on one more factor *i.e.* the available draft. *The consumption of fuel decreases as the depth of water between the bottom of the ship and the bottom of the river increases.* This is because, when the clear depth between the barge and the river reduces, an increased drag force is developed which increases the consumption of fuel. So much so, that the fuel consumption can be reduced by about 25%, if this clear space is increased from 0.6 m to 1.5 m or so. In order to maintain this minimum depth available, the ship will have to be loaded only to such an extent that a certain minimum clear space remains available between the bottom of the ship and the bottom of the river. Hence,

* Water transport is cheaper because on water, the coefficient of friction being small, one horse power can pull 4 tonnes ; while on road and rail, it can move approximately 0.15 tonne and 0.5 tonne, respectively.

the amount of loading of the barge is dependent and limited by the availability of sufficient water depth at the shallowest point in the route. From this discussion, it can be easily concluded that a certain minimum depth depending upon the type and extent of traffic is always required to be maintained in the river, so as to keep it fit for shipping or towing of barges. When this available depth or draft is not sufficient or if the channel width, channel alignment, locking time, current velocity, terminal facilities, etc. are not adequate, engineering measures are undertaken so as to increase the available draft and to improve other requirements. By undertaking more and more measures, and thereby spending more and more money on improving a waterway for navigation, we can make heavy and heavier transport feasible through that waterway. A compromise between the money invested on these technical works and corresponding advantages likely to be obtained by way of better and cheaper water transport, must be weighed against each other while working out a financial justification for such works. Besides the direct tangible benefits such as increased and cheaper communications for commerce and trade, etc. ; the intangible benefits such as recreational boating and better defence facilities for carrying military equipments, during war times, should also be given due importance, while working out the economic justification for such improvement works.

25.2. Various Requirements of Navigable Waterways

There is no rigidity about the requirements of a good navigable waterway, since it all depends upon the extent and type of traffic likely to pass through it. However, the various general requirements are enumerated below :

(1) Sufficient water depth is available so as to pass the more heavily loaded barges cheaply and economically.

(2) The width of the waterway is sufficiently more than the width of the tow itself.

(3) The radii of the bends should not be sharp and should be high enough to allow the maximum length of the ship to pass through them.

(4) The alignment of the waterway should be as straight as possible, because a highly irregular alignment increases the circuitry or length in excess of air-line distance which the barge tow must travel. The existing channels generally have a length about 50 per cent greater than the air-line distances.

(5) The flow velocities should not be high, as they may cause substantial reduction in the true speed for tows moving upstream and thereby increasing the time of transit and the cost of transport per kilometre. The speed of most of the barge tows in still water is of the order of 2.8 m/sec. The flow velocities of the order of 1 m/sec may, therefore, cause sufficient reduction in true speed (*i.e.* $2.8 - 1.0 = 1.8$ m/sec) and hence, should not exceed such a value.

(6) In order to minimise the transit time, the time required for the tow to pass through locks should be minimum. In certain cases, where the lock is not large enough to accept the entire tow, the tow is generally broken and taken through the lock in portions. This increases the time lost in locking, and thereby increasing the transit time and the cost of transport. Hence, sufficient sized locks should be ensured for economic and better transport.

(7) Efficient and adequate terminal facilities for unloading the barges for transferring the cargo effectively, must be ensured for economic and better navigation.

25.3. Various Measures Adopted for Achieving Navigability

There are three basic methods which are generally adopted for improving a river for navigation. They are :

(1) *Open channel methods*

(2) *Lock and dam arrangements* ; and

(3) *Canalisation.*

They are described below :

25.3.1. Open Channel Methods. In the open channel methods, the existing waterway is improved to such an extent as to make navigation possible. This improvement in natural waterway is possible only if the following conditions are satisfied :

(*i*) Sufficient discharge is available in the river throughout the year or at least for a reasonable portion of the year.

(*ii*) The existing river is having a satisfactory alignment without excessively sharp bends.

(*iii*) The river bed slope is reasonably flat so that the flow velocities are not excessive (*i.e.* they are within 1 m/sec or so).

(*iv*) The river width is not too small and is such that it can be improved economically for the modern barge tows.

(*v*) The material of the river bed and banks should permit satisfactory treatment by one or more of the open channel methods.

If the above requirements are approximately satisfied, the channel can be economically improved and made fit for navigation. But if the available conditions are far too short of requirements, open channel methods may prove to be highly uneconomical, and, therefore, should not be considered. However, these requirements and factors may be controlled to some extent by some suitable measures. Say for example, if the discharge in the river during lean periods is very low, while the average annual flow is adequate, reservoirs may be constructed so as to store water and augment the supplies during lean weather flows. Similarly, very sharp bends may be eliminated by cut off channels, provided the resulting channel slopes remain within limits.

The **various works and techniques** that may be involved in improving the channel by the Open channel methods are :

(*a*) *Constructing and Regulating the flow through Storage reservoirs.*

(*b*) *Excavation and Dredging.*

(*c*) *Contraction works.*

(*d*) *Bank stabilisation.*

(*e*) *Straightening the waterway by artificial cut offs.*

(*f*) *Removal of snag, debris and other obstructions.*

These techniques are generally required together as one of them may rarely provide the necessary required improvement. These techniques are described below :

(*a*) **Storage reservoirs.** The storage reservoirs generally store water during high flows and can release the required amount of water during lean-flows, so as to make downstream navigation possible even during periods of low weather flows. However, the construction and planning of storage reservoirs for navigation alone is not generally justified economically. Hence, reservoirs are mostly planned under Multipurpose

projects, where navigation may be one purpose of that project. Moreover, the storage reservoirs can augment low supplies for navigation, only if the reservoir is situated at the head of a relatively short navigable reach. This is because ; *as the distance from the reservoir to the navigable river-reach increases, reservoir-releases have to be increased so as to allow for transit losses due to seepage, evaporation, etc. The releases must also be made much in advance so as to allow for travel time to the navigable reach and their quantity has to be sufficient even after reduction due to channel storage.*

(*b*) **Excavation and dredging.** Huge amounts of excavations are generally required for clearing sand bars and filled channel sections in order to make it fit for navigation. Besides the basic initial excavations, continuous desilting and proper maintenance is required in order to keep the waterway fit for navigation. These excavations from the bed and banks of the waterway are generally carried out by *dredging* by means of dredgers. Three types of **dredgers** are generally used. They are :

(*i*) *Dipper dredgers.* They are merely floating power shovels and are used on small projects.

(*ii*) *Ladder dredgers.* They have an endless chain of buckets for bringing the excavated material upto the surface. The cuttings carried by buckets are discharged on a *belt conveyer* which are disposed of through a *stacker conveyor* at the rear of the dredger. Since the stacker conveyors (generally called *spoil stackers*) are limited in length to about 100 metres or so, ladder dredgers cannot be used when the excavated materials (*i.e.* spoils) are to be discharged at a considerable distance from the dredge.

(*iii*) *Suction dredgers.* In these dredgers, the cuttings and water are collected in suction pipes, and the mixture is then discharged by pumping through a pipe supported by floats (called spoil pipe) into the desired spoil area. A line diagram of this operational process is shown in Fig. 25.1.

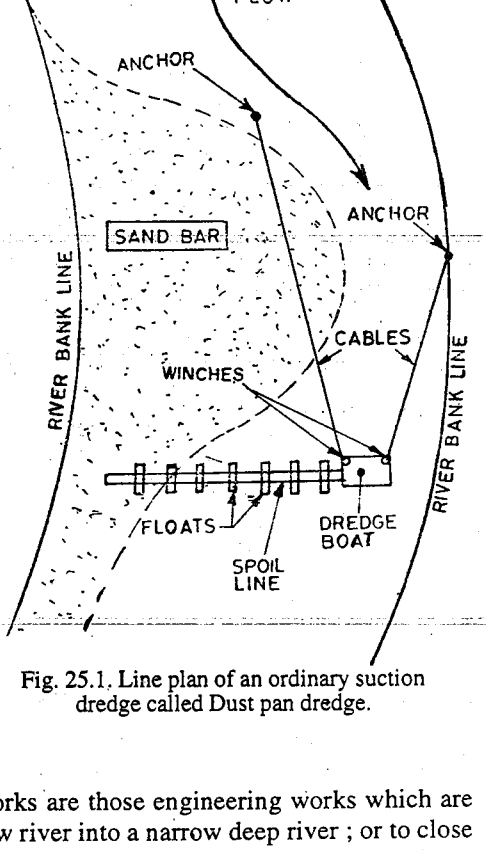


Fig. 25.1. Line plan of an ordinary suction dredge called Dust pan dredge.

A suction dredge cannot operate in rocky or boulder river reaches. The suction head of these dredges is provided with jets or rotating blades so as to cut or loosen the bed material, and also with suction openings through which the soil and water mixture enters into the suction pipe. These dredges can make cuts of about 10 m width through sand bars, and various such parallel cuts can be made in order to achieve a wider channel.

(*c*) **Contraction works.** Contraction works are those engineering works which are constructed in order to change a wide shallow river into a narrow deep river ; or to close off the river creeks (small branches) and thus to divert the entire water into the main

the material of bed and banks of a river is coarse grained with little shallow wide channel, or a number of coarse channels at low waters, will develop. Such situations may be corrected with the help of spurs or groynes. Under the process, rivers carrying huge sediment loads can be corrected with the help of properly placed permeable spurs called sal balli spurs made of sal ballies, driven at some suitable distance centre to centre in rows across the river current and braced at top. The function of permeable sal balli spurs is to slow the current and thus promote silting in the dyked area. The concentration of flow in the narrower section also encourages deepening of the channel. Several years are allowed for the effect of the structures to develop.

Similarly, the rivers carrying a little sediment load can be corrected by properly placed impermeable spurs or jetties which shall divert the flow, thereby confining the entire water in a smaller width and thus deepening the same.

(*d*) **Bank stabilisation.** A good navigable channel must have stable banks. When the river banks are not stable and start caving, the river starts meandering, creating bends, which may obstruct the path of longer barge tows. Moreover, scouring at concave banks and silting at convex banks take place due to meandering (as explained in chapter 8). Hence at bends, sufficient depth will prevail at least near the concave side. But the tangents, *i.e.* the crossings joining the two successive bends, will definitely develop shallower channels with cross bars by the deposition of sediment scoured from the upstream bend. *It is in these crossings that the controlling depths for navigation occur.*

Spurs or groynes, when suitably and intelligently placed, may prove to be useful in bank stabilisation ; because a spur placed along the concave bank shall promote silting. Banks may be protected more easily by *pitching* or by *rivetments*. The entire concave bank is generally protected by pitching. The loosely dumped stone called apron or riprap is generally used, and it is extended from top of bank to beyond the toe of the underwater slope. This extension of rivetment in the bed is essential so as to avoid the failure of rivetment due to scour and consequent undermining of the underwater edge of the rivetment. The rivetment must be flexible so as to adept itself to the surface on which it is placed. Moreover, the rivetment must be relatively impervious so as to avoid, the washing of fines through it. It must also be strong enough to resist the flow currents. Various types of rivetments are used. Concrete mattresses in the form of concrete blocks placed in wire meshes may sometimes be used, when ordinary stone dumping over a graded filter is not provided due to non-availability of stone in the nearby areas. Uncompacted asphalt paving is also finding a use in developed countries, and is under serious investigations. Compacted asphalt paving and monolithic concrete paving are not generally used, as they are liable to be cracked and damaged by uplift pressures.

(*e*) **Straightening by artificial cut-off.** It was explained in chapter 8, as to how and when a cut off develops in a meandering river. Since the development of a cut-off eliminates sharp bends which are undesirable for navigation, artificial cut-offs may sometimes be used advantageously. A pilot cut is made and allowed to develop (Fig. 25.2). These cut offs have been used with

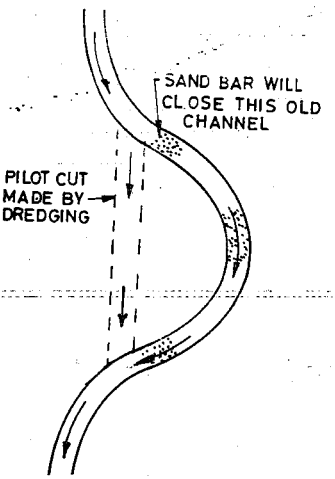


Fig. 25.2.
Development of a cut off.

success at various places, provided the banks are stabilised to avoid future caving and meandering.

(*f*) **Removal of snag, debris and other obstructions.** Presence of debris, trees, isolated rocks, and other obstructions, not only pose a direct hazard to the barge tows, but also promote the formation of sand bars. They must, therefore, be removed effectively in order to ensure safe and economical navigation. Different methods and equipments may be used in different cases, depending upon the circumstances of each case. Tractors, winches, derrick barges, explosives, etc. may be required in the process of clearing the waterway of these obstructions.

25.3.2. Lock and Dam Arrangements. The arrangement consists of dams which create a series of slack water pools through which the traffic can move with locks to lift the vessels from one pool to the next. Lock and dam construction may be adopted where existing site conditions are not favourable for adopting open channel methods described earlier. *This arrangement is a second choice to open channel methods.* In this arrangement, water is required for lockages, sanitary releases, evaporation, percolation, etc. This requirement of water is much less than that required for open channel procedures. Hence, when the available water is less, these arrangements may have to be adopted.

The slack water pools behind the dams, will submerge the rapids and channel bends, and thus overcoming those problems. Further, because of their relatively large areas of cross-sections, the velocities in these pools shall be low enough as to cause lesser reduction in true speed of the barge tow moving upstream.

Lock and dam arrangements are suitable only on rivers bringing only a little sediment load. Because, highly silt laden river-waters shall fill up the pools rapidly. Moreover, suitable sites for construction of small dams must be available for providing such arrangements. Further detailed descriptions of these arrangements are beyond the scope of this book.

25.3.3. Canalisation. A totally new channel-cut is provided artificially around an otherwise impassable obstruction or between two navigable rivers. Such a cut is generally economical only when a short length of new channel opens a large length of existing waterways. Construction of a new channel connection between two existing waterways is also sometimes adopted, so as to ensure a continuous traffic way. However, canalisation is a costly process, as the per kilometre cost of canal, capable of passing modern barge tows, is normally very high, and are adopted when very short lengths are required.

25.4. India's Navigable Waterways

Unfortunately, there is no river in India running from North to South or *vice versa.* Most of the Indian rivers run from West to East or from East to West. This is the major handicap in the development of navigation in this country. Although there are about 15,000 kilometres length of navigable waterways in India, but most of them are in very bad conditions due to silting and poor maintenance. Most of these waterways are fit only for boating except for about 2,500 kilometres length, in which the steamers can ply. Out of the total navigable lengths of about 15,000 km ; about 8000 km is on rivers, and 7000 km is on canals and backwaters. Most of these navigable waterways exist in Ganga and Brahmaputra rivers and their tributaries. On account of its inherent advantages, the water transport has been widely developed in various countries (especially in U.S.A.), but a lot of work remains to be done in India. Let us hope that as soon as our Government finds some free investment, the same will be invested in improving our navigable

water-ways. Although the initial investment may be high, but the advantages obtained shall be tremendous, as it will not only provide us cheap transport, but would relieve the burden upon our rail and road transport.

PROBLEMS

1. (*a*) What do you mean by 'River navigation'? What is the importance of river navigation in modern system of transport ?

(*b*) What are the requirements of navigable waterways, and what measures are adopted to satisfy these requirements ?

2. (*a*) Discuss the importance and necessity of improving India's waterways.

(*b*) Enumerate the different methods which are adopted for achieving navigability in a river. Discuss one of these methods in details. Also discuss the pros and cons of different methods and their use for a particular project.

3. Write a descriptive note on 'River navigation'.

26

Tank Irrigation

26.1. Definition and General Introduction

Tank irrigation may be defined as the *storage irrigation scheme*, which utilises the water stored on the upstream side of a smaller* earth dam, called a *bund*. These earthen bund *reservoirs* are, thus, infact, called as *tanks*, specifically in South India, where such works are very common. This terminology is, therefore, limited to India only.

There is, thus, no technical difference between a 'reservoir' and a 'tank', except that a large-sized tank will be termed as a reservoir. Moreover, a reservoir will generally be formed by dams of any material (such as masonry dam, concrete dam, or earth dam, etc.) ; whereas, a tank is generally said to be formed by earth dams only (or strictly speaking by earthen bunds). These earthen bunds, spanning across the streams, are called *tank bunds* or *tank banks*.

Most of the existing tanks of south India possess a *maximum* depth of 4.5 m, while a few are as deep as 7.5 to 9 m, and only a few exceptional ones exceed 11 m in depth. When the depth of the tank exceeds 12 m or so, the tank is generally referred to as a *reservoir*.

Like all earth dams, tank bunds are generally provided with *sluices* or *outlets* for discharging water from the tank for irrigation or other purposes. These *tank sluices* may be pipes or rectangular or arched openings passing near the base of the bund and through the body of the bund, and carrying the water into the downstream channel below the bund or transporting at distances where required, through pipes or canals. Sometimes, the *supply sluices* may not be passed through the body of the bund, and may be carried adjacent to it through some hill side at one end of the bund.

Similarly, as in the case of all dam reservoir projects, tanks are provided with arrangements for spilling away the excess surplus water that may enter into the tank, so as to avoid over-topping of the tank bund. These *surplus escape arrangements* may be in the form of a *surplus escape weir*, provided in the body or at one end of the tank bund, or some other arrangement like a syphon spillway may be provided as is done in the case of earth dam projects. The surplus escape weir is a masonry weir (compared to an ogee spillway in an earth dam) with its top *i.e.* crest level at equal to *Full tank level* (F.T.L.). When tank is full upto FTL, and extra water comes in, it is discharged over the surplus escape weir. The length or capacity of this surplus escape weir will be so designed that water level in the tank does never exceed the *maximum water level* (M.W.L.). The top of the tank bund will be kept at a level, so as to provide a suitable free-board above this M.W.L.

Since the surplus escape weir is a masonry weir, it will have to be properly connected to the earthen bund by suitably designed bank connections.

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* Generally less than 12 m or so in height.

26.2. Isolated Tanks and Tanks in Series

Most of the existing small sized tanks of south India form part of *groups of tanks*, which are connected together in series, such that any tank either receives the surplus water of the upper tank(s), or sends its own surplus into some lower tank(s), or do both. However, when a tank neither receives water from an upper tank nor discharges its own surplus into a lower tank, it is called an *isolated tank*. There do exist some isolated tanks also in South India.

Figure 26.1. shows a typical group of 15 tanks numbered 1 to 15, and an isolated tank A. It is evident that *considerable economy of water can be obtained from the system of grouping*, because the surplus water of the each tank and also the drainage of its wet cultivation are caught up by the next lower tank. Each tank of the group takes a share in the yield of the whole catchment above it, which can be classified as follows :

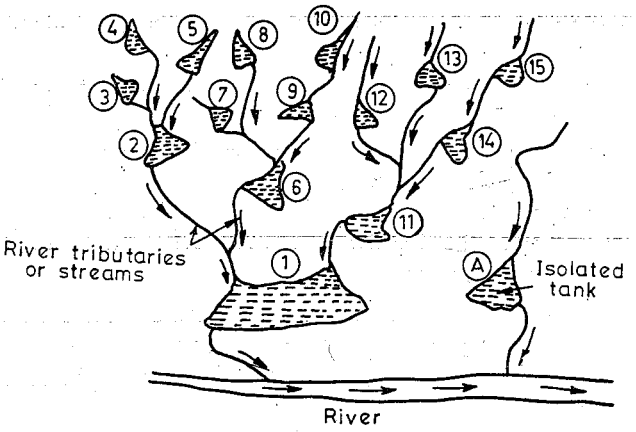


Fig. 26.1. Fig. showing isolated tanks and tanks grouped in series.

(*i*) the *free catchment*, which is the catchment area which only drains into the tank under consideration ; and

(*ii*) *the combined catchment*, which is the area of the whole catchment above the tank under consideration.

The difference between the combined and the free catchment, thus, gives the area of catchment intercepted by upper tanks. The whole catchment of the highest tank, on each drainage, shall be its free catchment. Moreover, each tank will receive the whole run- off from its free catchment ; but from the remainder of its catchment it will receive only the balance runoff which remains after the upper tanks have been filled.

One of the disadvantage of grouping of tanks, however, is that, if a breach occurs in an upper tank, it exposes all the tanks in the series below, to the risk of similar failures. Hence, while making repairs or doing restoration works in such groups of tanks, it is necessary to consider each system of tanks collectively as a whole, and to carry out repairs from the upper end of each group. The extent of danger to lower tanks due to breach of an upper tank, however, depends upon their relative capacities. For example, if a small upper tank breaches and passes its storage into lower large-capacity tank, it may pose little danger even if the breach may completely empty the small tank. However, the danger will be imminent if the case is opposite, and the large sized upper tank breaches and passes its storage into the lower small sized tank.

The provision of suitable *breaching sections*[*] in tanks forming part of a group is, therefore, considered as a very desirable precaution against their failure by breaching. This is because, a breach occurring in a tank bund at a well selected point (at the place of breaching section) will not empty the tank, and indeed frequently does little more than supplement its surplusing power to a reasonable degree to the greater security both of the tank itself and those below it in the group.

26.3. Capacity of Water Spread of a Tank

The *gross capacity of a tank* may be defined as the cubic content of water stored in the tank upto F.T.L. The *effective capacity of a tank*, will, however, be the cubic content of water stored between F.T.L. and the bottom or sill level of the lowest supply sluice.

These storage capacities can be computed easily by using the contour plan of the area of the water-spread ; the total capacity being the sum of the capacities between successive contours. The smaller the contour interval (Δh), the more accurate the capacity computation will be. This is because, if A_1 and A_2 represent the areas enclosed between two successive contours, then the cubic content between these contours is roughly taken as $\frac{A_1 + A_2}{2} (\Delta h)$.

The summation of all cubic contents between the successive contours will be the required storage capacity of the tank.

When the contour plan is not available, and only the area of the tank at F.T.L. is known, then the effective cubic content of the tank may be roughly computed as follows

The area of tank at F.T.L. is multiplied by one- third of the depth from this level (F.T.L.) to the deep bed of the tank, or the level of the sill of the lowest sluice, whichever is higher of the two.

If the area (A_1) of the tank bed at the level of the sill of the lowest sluice, and the area (A_2) of the tank at F.T.L. are known even roughly, then the effective cubic content of the tank may be computed as $\frac{1}{3} [A_1 + A_2 + A_1 A_2] \times h$, where h is the height or difference in elevations between F.T.L. and sill level of the lowest sluice. This formula is based on the assumption that the water stored is in the shape of a frustum of a cone.

These principles on which the storage capacity of a reservoir is computed, were also discussed earlier under article 18.2.

26.4. Designing the Section of the Tank Bund

The tank bund, as you know, is nothing but a small-sized earth dam ; and hence, strictly speaking, its design and construction should be carried out in accordance with the principles applicable to earth dams (already discussed in details in chapter 20).

In accordance with these very principles, the tank bunds may be of three types ; *i.e.*

Type A : *Homogeneous embankment type* (Fig. 20.1) ;

Type B : *Zoned embankment type* (Fig. 20.2) ; and

Type C : *Diaphragm type* (Fig. 20.3).

Most of the tank bunds of South India belong to type A, and they have been constructed with the soils excavated from pits in the immediate vicinity of each bund, and carried by head load to the bund.

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* A 'breaching section' is a length kept lower and weaker than the remainder of the bound, so as to localise a breach in that length only.

As discussed in chapter 20, the large modern earth dams are designed after doing a lot of mathematical calculations for seepage and stability analysis, and keeping into consideration the latest advancements made in the field of soil mechanics. However, for small earthen bunds, generally no such calculations are carried out, and the sections are designed on the basis of practical considerations and experience derived from success and failure of similar works and of their working conditions.

The commonly adopted standards used for fixing the dimensions of tank bunds, in South India, are given in table 26.1.

Table 26.1. Common Dimensions of Tank Bunds

S. No.	*Depth of deep bed below F.T.L. i.e. maximum water depth in metres*	*Free board (in metres)*	*Width of top of bund (in metres)*
1.	1.5 to 3.0	0.9	1.2
2.	3.0 to 4.5	1.2	1.5
3.	4.5 to 6.0	1.5	1.8
4.	over 6.0	1.8	2.7

In favourable soils, such as red and white gravel, red and black loams, etc., the side slopes of the bund may be kept as $\frac{1}{2} : 1$ ($\frac{1}{2} H : 1 V$) for smaller tanks with water depths not exceeding 2.5 m, and 2 : 1 for larger ones up to about 5 m in depth. In light sandy, or black cotton, or clayey soils, however, the slopes may be kept between 2 : 1 to $2\frac{1}{2} : 1$.

The upstream face of the tank bund is generally rivetted with stone apron or riprap (Fig. 26.2) so as to protect it against erosion, and if this is done, then the upstream slope

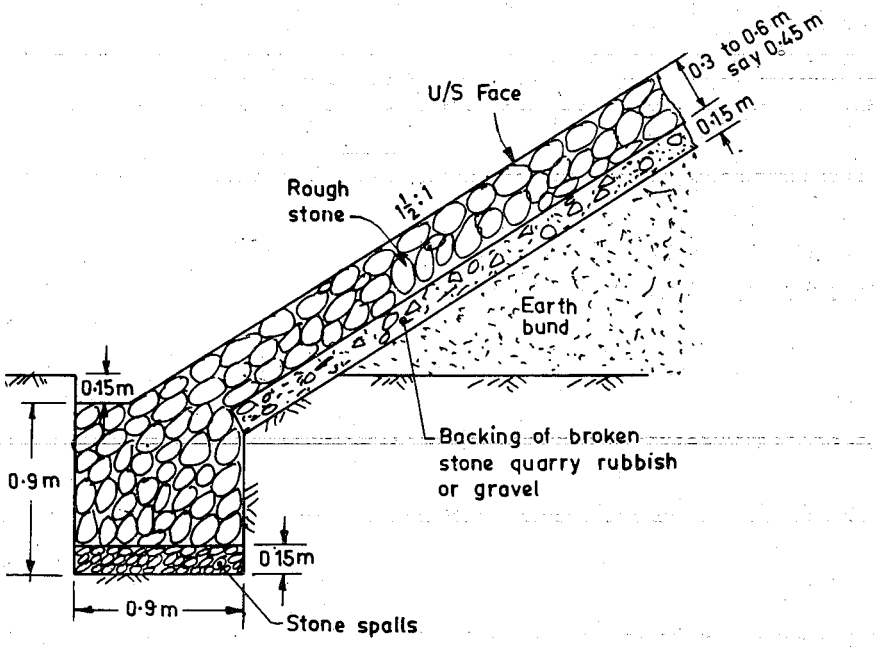


Fig. 26.2. Upstream Revetment of Tank Bunds.

is generally adopted is $1\frac{1}{2}$: 1 even upto 6 m depth. For inferior soils, or greater depths, however, the rivetted slope may be made flatter, say 2 : 1.

In this way, for average cases, a $1\frac{1}{2}$: 1 slope will generally be adopted for upstream face, and 2 : 1 slope for downstream face.

This practice is *contrary* to the standard recommendations adopted in various countries, where the upstream slope, even when revetted, is kept flatter than the downstream slope because of the soil being saturated. There are, however, thousands of tanks in Tamilnadu with water slopes of $1\frac{1}{2}$: 1, and failure by slipping of this slope is rare ; and hence, in normal cases, the local practice in this respect can be easily adopted. In very small tanks, and in case the upstream slope is heavily revetted, upstream faces have been given 1 : 1 or even steeper slopes in actual practice, but such steeper slopes are not recommended.

26.5. The Tank Weirs or Surplus Escape Weirs and Their Design Principles.

26.5.1. Definition. As stated earlier, the excess surplus water is spilled from a tank, into the downstream channel, so as to avoid the rise of water in the tank above the M.W.L. In fact, the water will generally start spilling over the crest of this escape weir, as and when it rises above F.T.L. ; and the discharging capacity of this weir will be designed so as to pass the full maximum flood discharge (likely to enter the tank) with a depth over the weir equal to the difference between F.T.L. and M.W.L.

Although the effective storage capacity of a tank is limited by F.T.L., but the area submerged by the tank bund and revetment are all dependent on M.W.L. And hence, in order to restrict the dimensions of these, it is desirable to keep the difference between F.T.L. and M.W.L. to a smaller value. But, on the other hand, the smaller is this difference, the longer will be the surplus escape required in order to enable it to pass the given discharge. Hence, the difference (H) between F.T.L. and M.W.L. is fixed on a compromise basis, in each particular project, so as to obtain overall economy and efficiency. In small and medium sized tanks, the usual difference between F.T.L. and M.W.L. is kept from 0.3 to 0.6 metre, and it is rarely allowed to exceed 0.9 metre.

26.5.2. Types of Tank Weirs and Their Cross-sections. Tank escape weirs are similar to river-weirs (*i.e.* diversion weirs or anicuts), and hence, they may be classified into the following three general types :

Type A : *Masonry weirs with a vertical drop ;*

Type B : *Rockfill weirs with a sloping apron* ; and

Type C : *Masonry weirs with a sloping masonary apron (glacis).*

These three types of river weirs have already been discussed in article 9.3.1.

Besides these three important types of river weirs, which are used as tank weirs also, the fourth type of a tank weir which is a combination of type A and type C, may also be used. In such a *D type weir,* a number of vertical steps are made (as in the case of a stepped fall) instead of providing a horizontal or sloping downstream apron. Such weirs of type D are called *weirs with stepped aprons.* The A and D type of weirs are most widely adopted, although every type of weir has been used in South Indian tanks.

The A type of weir *i.e.* weir with a masonry pucca horizontal floor, may sometimes by provided with depressed floors, so as to provide sufficient water-cushion for absorbing the impact of high water falls. Typical sections of the two sub- types of A type, and

D type of weirs, and the conditions under which each is adopted are shown in Figs. 26.3 (*a*), 26.3 (*b*), and 26.4 respectively.

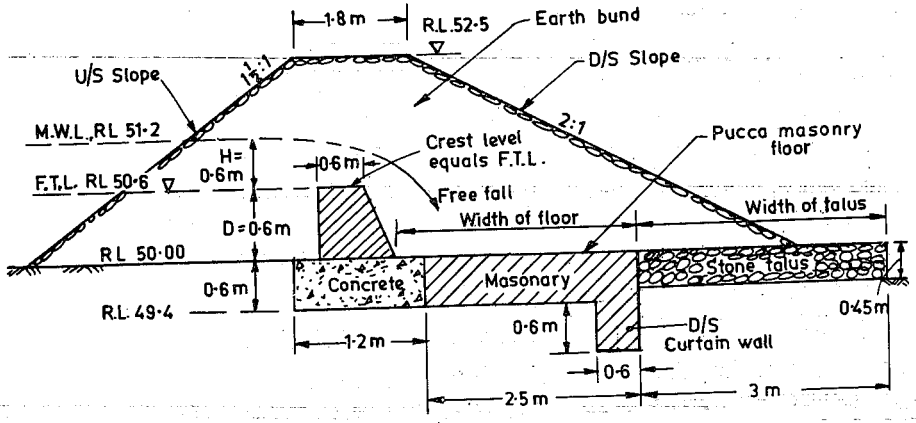


Fig. 26.3 (*a*) A typical masonry tank weir with a vertical drop, with a horizontal undepressed floor, suitable for low drops of 0.6 to 0.9 metre or for even higher drops when the soil is a rock.

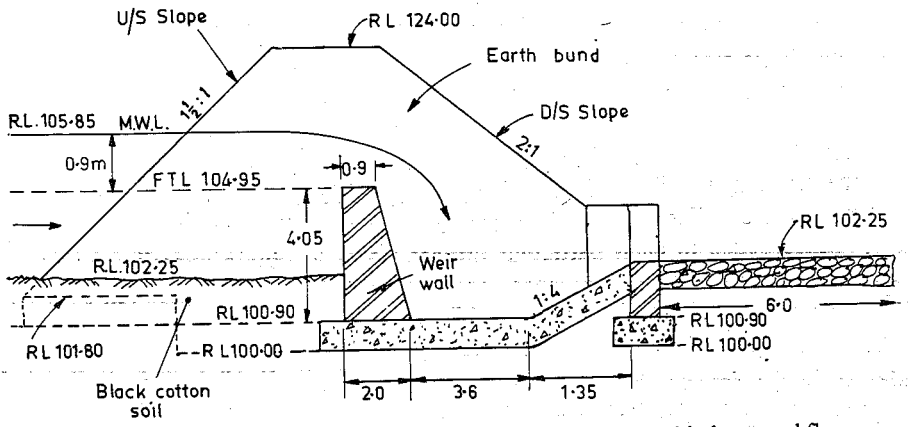


Fig. 26.3 (*b*) A typical masonry tank weir with a vertical drop with depressed floor, suitable for high drops (say 2.5 m upward) or in soft soils but only slightly pervious.

The general principles and considerations which guide the design of tank weir sections, are the same as in the case of river weirs founded on *rocks* or *loose pervious* or *impervious soils* But in case of tanks, the conditions regarding the *nature of foundation soil* and the *working conditions* are more varied than in the case of river weirs.

Say for example, the river weirs or anicuts are generally founded on soft sandy soils, whereas in tank weirs, almost every class of soil is occasionally to be built on, except this rarely met foundation sand.

Similarly, the tail water conditions in anicuts differ from those in tank weirs. This is because, the tail water backing up during floods and affording some protection to the talus and to the soil at the downstream toe of the apron, are absent in the majority of tank weirs ; and, therefore, the tank weirs built on soft soil would, other things being

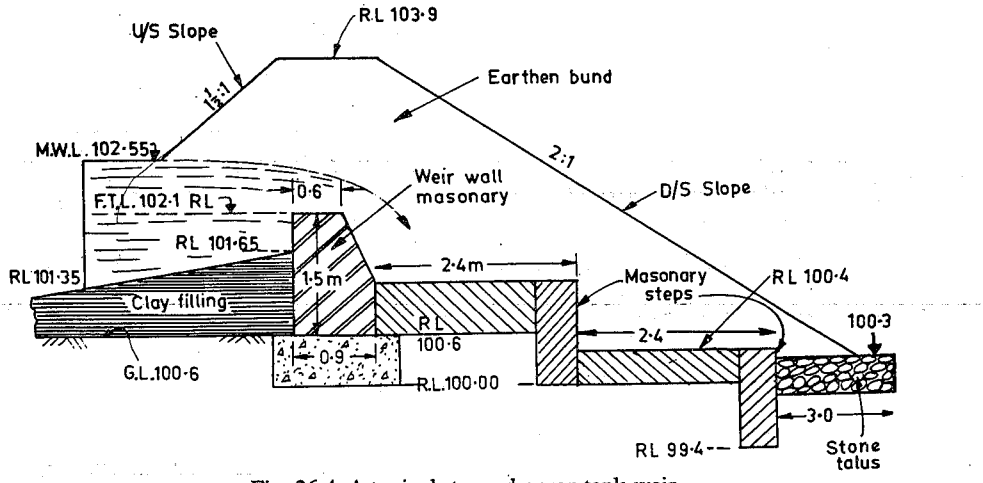


Fig. 26.4. A typical stepped-apron tank-weir, suitable for sites on soft soil when the drop exceeds 0.9 metre.

equal, be more exposed to damage by scour at the downstream toe of the apron and retrogression of levels than river weirs.

On the other hand, the depth of water passing over the crests of tank weirs is generally far less than that over anicuts, and in the case of very many tanks, it is only, on an average, for a few days each year, that any flow at all passes over the weir, and while the duration of continuous heavy flow is frequently limited to a maximum of two to three days. Tank weirs are, thus, usually worked, far less heavily and far less continuously than anicuts, and this fact may be duly considered while designing them.

Empirical Formulas for Determining Width of Floors of Tank Weirs. The width of the horizontal floors of type *A* and *D* weirs, from the foot of the drop wall to the downstream edge of the floor should never be less than 2 $(D + H)$ where D is the height of the drop wall, and H is the maximum water head over the wall. In important works, this width may be increased to 3 $(D + H)$. The rough stone apron forming a *talus* below the last curtain wall may be of varying widths depending on the nature of the soil and the velocity and annual probable quantity and intensity of runoff ; it would generally vary from 2.5 $(D + H)$ to 5 $(D + H)$ according to varying conditions.

26.5.3. Length of the Tank Weir. In order to determine the length of the escape weir, it is, first of all, necessary to determine the *maximum flood discharge* that may enter into a tank, after it is filled up to full tank level. This peak discharge that may come from the free catchment of a tank can be fairly estimated by using the empirical formulas applicable to the given region, as discussed earlier under article 7.9.2. Generally, in South India, all tank weirs have been designed on the basis of Ryve's formula (Refer equation 7.49). The formula is directly applicable for free catchments in all isolated tanks. However, for combined catchment of a tank forming a constituent unit of a group, the following modified formula is used for calculating peak discharge :

$$Q_p = C_1 \cdot A^{2/3} - c_1 a^{2/3} \qquad ...(26.1)$$

where C_1 is the coefficient in Ryve's formula (Refer Table 7.45)

A is the area of the combined catchment in sq. kilometres

c_1 is the coefficient from $\frac{1}{5}$th to $\frac{1}{3}$rd of C_1 and

a is the area in sq. kilometres of the catchment intercepted by upper tanks.

This formula is purely empirical, and should only be applied when the upper tanks are provided with adequate surplus works, and kept in good state of repairs.

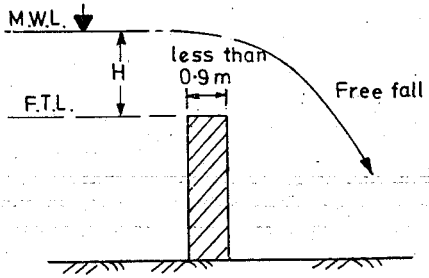
After the peak discharge is worked out and decided upon, the length of the weir can be worked out by using the appropriate discharge formula applicable to the weir type adopted in the project. In general, the discharge over a broad crested free weir and without any velocity of approach, is given as :

$$Q = C \cdot L \cdot H^{3/2} \qquad ...(26.2)$$

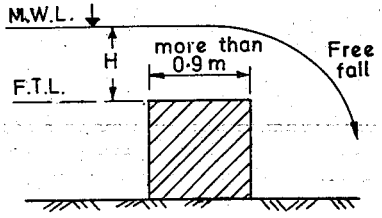
where L = the length of the weir ;

H = the head of water over the weir (*i.e.* the difference between M.W.L. and F.T.L.), and

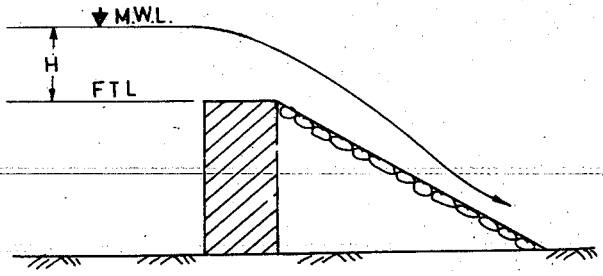
C = a constant, the common values of which for three commonly used types of weirs in South India, are shown in Fig. 26.5.



(*i*) Weirs with crests up to 0.9 m wide.



(*ii*) Weirs with crests over 0.9 m wide or weirs provided with dam-stones.*



(*iii*) Rough stone sloping escapes.

Fig. 26.5. Different types of Tank Weirs and the values of their coefficients in discharge formulas.

*A common practice which has been resorted to for centuries in South India is to fix **damstones** in the crest wall of the weir. these stones are generally about 15 cm × 15 cm in section, and 0.75 to 0.9 m

high, built into the masonry weir wall at about 0.45 to 0.6 m clear interval ; projecting above the weir crest to a height of 0.3 to 0.6 metre, their tops being generally at M.W.L.

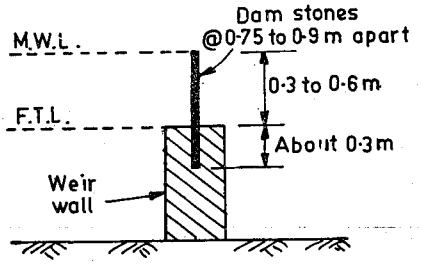


Fig. 26.6. Dam-stones.

The space between these stones are blocked up with clay and turf by cultivators, with the result that water can be held up in the tank up to M.W.L., before any surplusing takes place. When the tank receives more water, which just causes the water level in the tank to rise above M.W.L., the water over-tops the turf weir, which is rapidly washed away, and the surplus weir starts functioning with full head (H), without M.W.L. exceeding to any appreciable extent.

Whenever this occurs, all the water stored in the tank above the weir crest level, must however, be passed off before the turf dam can be rebuilt.

The dam stones cause obstruction to the discharge, and when they are installed, the length of the weir must be correspondingly increased, *i.e.* constant (C) in weir discharge formula reduced from 1.84 to 1.66.

Sometimes, instead of damstones, shutters may be provided between F.T.L. and M.W.L., so that they can be lowered and raised as and when needed, thus serving the purpose of giving full pondage up to M.W.L. before the worst flood arrives for surplus disposal. They may, however, create operational problems, and necessitate special establishment.

If no damstones or shutters are provided above F.T.L., and flood water starts spilling over the surplus work immediately after the tank water level exceeds F.T.L., then *strictly speaking*, the computed peak flood discharge should not be used to directly work out the length of the weir by the above simple and straight formulas. This is because of the fact that certain flood discharge will be absorbed between F.T.L. and M.W.L. This *flood absorption capacity* of a tank is, however, considered only in very big tanks or in large-sized tanks, and this is kept unaccounted as a further measure of safety in such works. A method of computing the allowance which may be made for the absorption capacity of big tanks has been worked out by Capt. Garret, and is given in his paper on the "General Theory of the Storage Capacity and Flood Regulation of Reservoirs" published in 1912 by the Government of India. The same may be referred to if needed along with its original tables given in the same paper.

26.5.4. Choice of a Suitable Site for Locating a Tank Weir. As pointed out earlier, a surplus weir can be located either in the body of the tank bund or at one end of it. When a saddle, disconnected with the tank bund is available, it must be preferred for locating the surplus weir, as compared to providing it within the bund body, but even in such a case, it will be necessary to locate it at one flank of the bund and connected with it.

If all the cultivation is on one side of the stream which is dammed by the bund, then it is preferable to locate the surplus weir on the other flank ; otherwise it would be necessary in many cases, to carry the distributary channels across the surplus channel,

and there may also be danger of cultivation being damaged by the surplus water when the tank surpluses freely.

Besides these considerations, the most favourable sites for surplus weirs are at places ;

(*i*) where the N.S.L. along the line of the weir approximately equals F.T.L.

(*ii*) when the foundation soil is hard both on the weir site and along the runoff channel, and where the ground contours are such as to give clear approach to the weir site and a suitable course for the surplus water.

26.6. Tank Outlets or Tank Sluices and their Design Principles

As stated earlier, a *tank sluice* is an opening in the form of a culvert or a pipe running through or under the tank bund, and supplying water from the tank to the distributary channel below, to meet the irrigation or other water requirements, as and when needed. Suitable wing walls and other bank connections are also provided as required at the head and tail end of the culvert.

In ordinary medium-sized sluices, masonry culverts with or without arch roofs are generally constructed (Refer Fig. 26.7). The size of the culvert (*i.e.* its cross-section) will depend upon the maximum quantity of water (Q)* it is required to convey, but in no case should be less than 0.6 metre wide and 0.75 metre high, so as to allow a man to enter it for examination and repairs or removal of obstructions. The size of the barrel should also be such as to limit the velocity through the sluice barrel to a maximum of 4.5 m/sec*, under the condition of plug hole being fully open and with the water at full tank level.

In case of very small sluices, earthen ware or cement or cast iron pipes may be used to take the place of masonry culverts. (Refer Fig. 26.8). Such sluices are called *pipe sluices*. They are generally not adopted in tank bunds, where the depth below F.T.L. exceeds 2.5 metre or so. This is because, in such cases, the earthen ware pipes may get fractured, or leakage through their joints may take place, resulting in a breach, as the pipes can neither be examined nor repaired easily without cutting open the bund. Moreover, these pipes and especially the cast iron pipes are rarely found to be economical compared to masonry sluices.

Besides the above types of tank sluices, one more type of sluice *i.e. a sluice with a tower head* (Refer Fig. 26.9) is also sometimes provided. This form is sometimes found more economical than the types previously referred to, owing to the saving obtained by avoiding the heavy wing walls of the previous type. Moreover, here there is lesser danger of failure by cracking and bulging which frequently occurs in the wings of the other type. But, on the other hand, with such a sluice, additional length of the barrel is required. Tower-heads or wells are generally placed in the water slope of the bund, as thereby, the expanse of a bridge or causeway leading from the bank to the regulation

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*$V = \frac{Q}{A} = \frac{\text{Discharge}}{\text{Area of barrel}} = Cd \cdot \sqrt{2g \cdot h}$

where h is the height of the F.T.L. above the sill level of the sluice.

Cd = 0.63 for free fall.

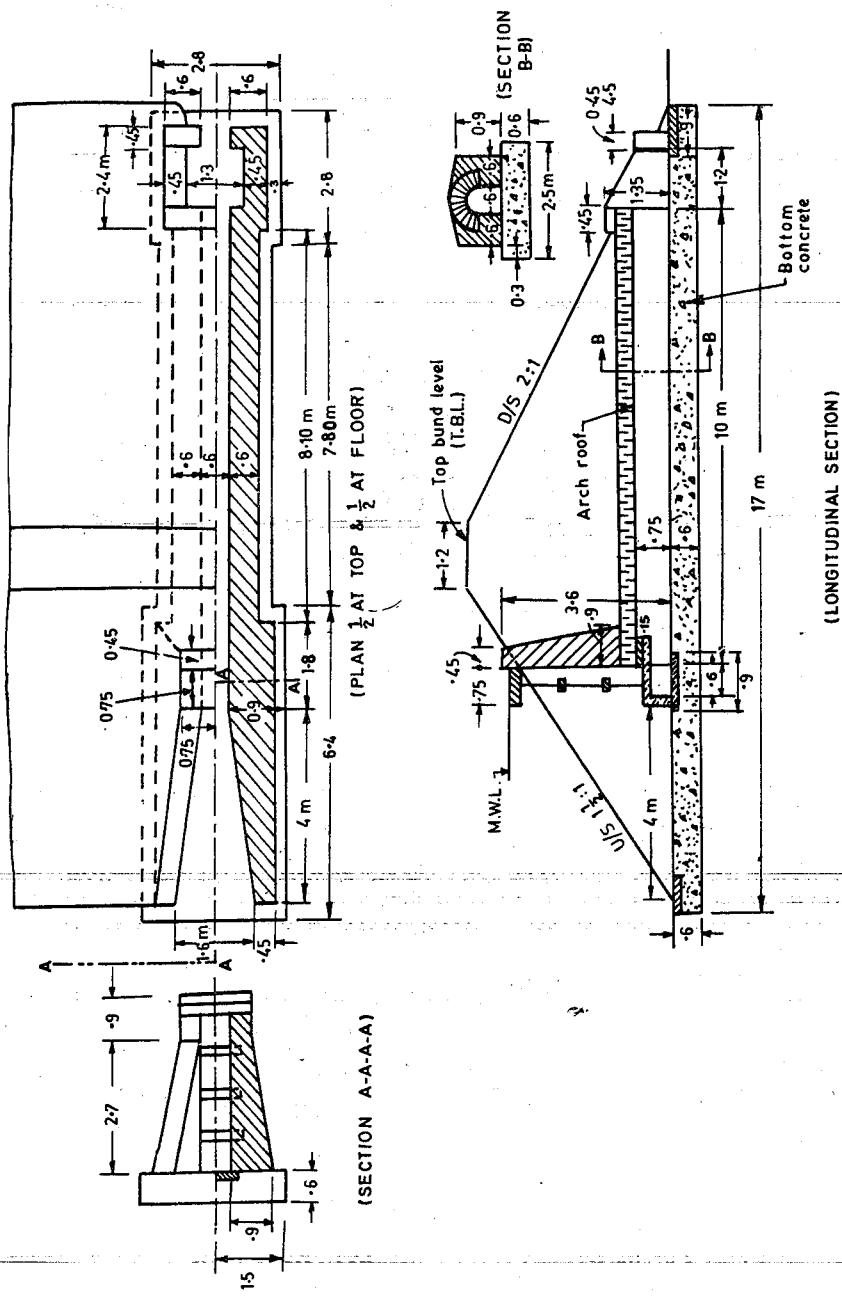


Fig. 26.7. Masonry supply sluice for a medium-sized tank.

platform is saved. But this has the disadvantage of making it impossible, while the tank has water, to have access to the portion of the barrel upstream of the well.

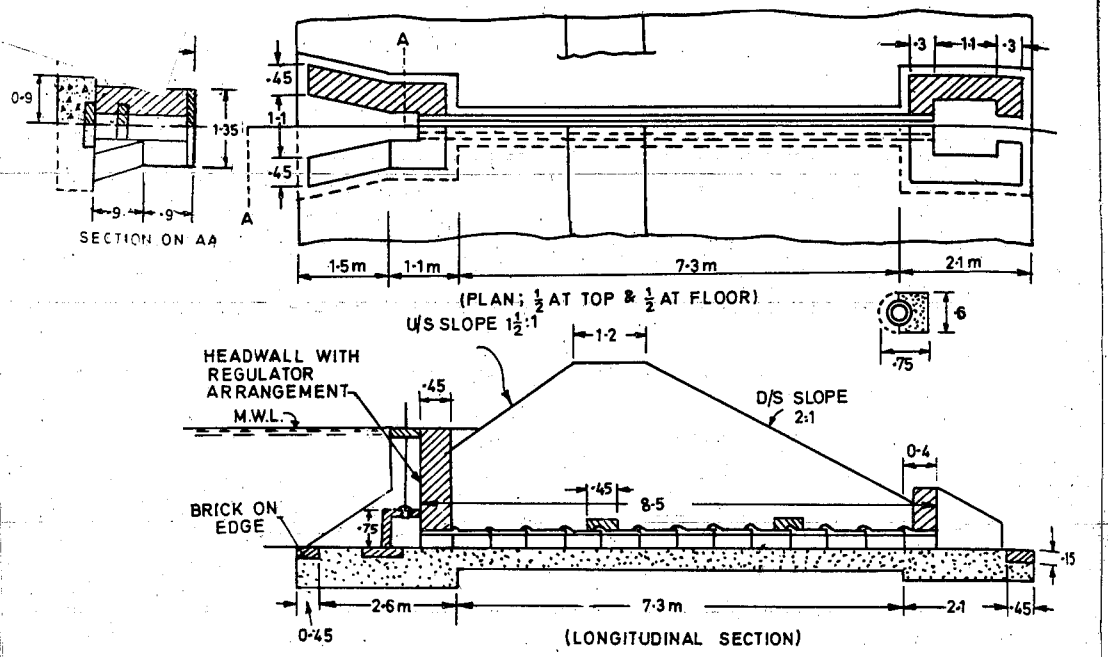


Fig. 26.8. Earthern ware pipe-sluice for a very small tank.

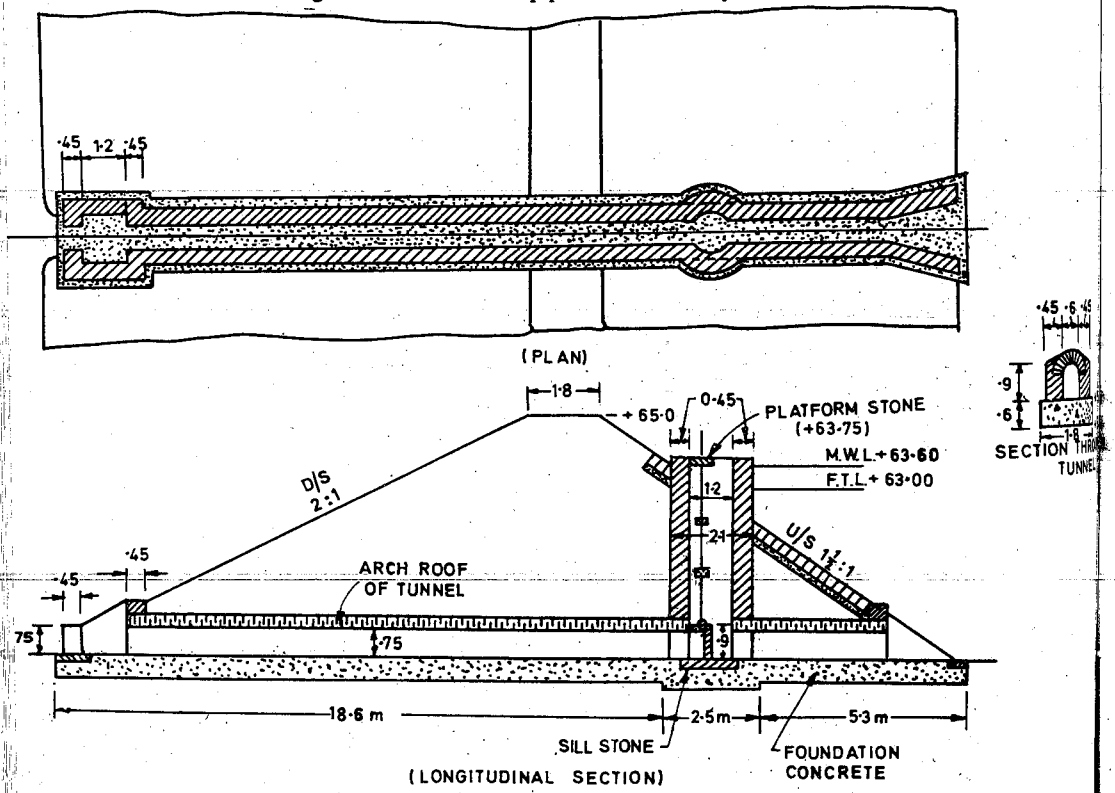


Fig. 26.9. Tank sluice with a tower-head.

26.6.1. Method of Regulation of Tank Sluices. The generally used vents for medium-sized tank sluices consist of one or more circular orifices of suitable size made in a horizontal stone, and the area of the opening is regulated by a long coned plug, the wider end of which is of the same diameter (allowing for clearance) as the vent (Refer Fig. 26.10). The vent is termed as the *plug-hole*. As the plug is raised, the hole or orifice is opened more and more, and when the plug is raised altogether clear of the plug hole by an amount not less than the diameter of the hole, the full discharge from the orifice will pass. This arrangement is both simple and economical, and affords an easily adjustable means of regulation to suit the varying conditions of water level which are inseparable from the reservoirs, and this arrangement is, thus, more suitable for tank sluices than the flat shutters which are generally used for canal sluices.

Details of plugs for tank-sluices are shown in Fig. 26.10, the standard sizes being 100, 150, 200, 250 and 300 mm dia, this being the size of the hole in the plug stone, the largest size of the cone part of the plug being about 6 mm less ; the width of the

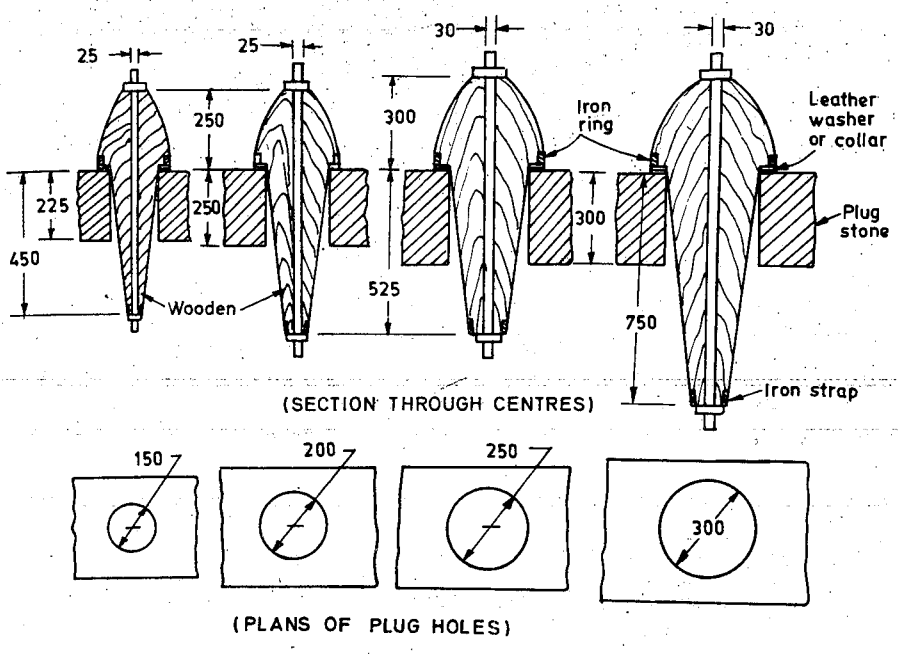


Fig. 26.10. Typical standard sized plugs for tank sluices.

seating should be from 25 mm to 30 mm. The cones should have a taper of one to four, and be made of hard wood ; the smaller end of cone and the greatest dia. of the plug should be cut cylindrical to take countersunk iron straps which should be shrunk on to prevent the plug from splitting. The lifting rod should be round iron, and secured by a split cotter pin at each end of the plug. When the rod is long, it should be passed through holes in guide beams fixed at intervals. The top end of the rod passes through a hole in the regulation platform ; it should be flat iron with holes at about 75 mm interval through which an iron pin is passed to keep the plug suspended at whatever height it gives suitable elevation.

When the computed size of a tank sluice or plug hole is more than 250 to 300 mm, in diameter, then it is generally advisable to provide two or more holes and plugs in the plug platform stone ; while for deep sluices, say over 7.5 m depth, two or more plug stones at different levels, each with separated plug holes, provide greater facility for easy regulation, and hence adopted.

The size of the orifices in the plug stone are generally calculated so as to pass the full supply with vents fully open and with 0.3 m of water standing over the plug stone platform. For regulation below this level, a rectangular vertical vent is provided which is closed by a shutter, which may be wooden but is generally a slab of stone. This vent is made of sufficient size to pass full supply with 0.15 m head. This shutter is brought into use only when the water in the tank is less than 1.2 m over the floor of the sluice; and at other times, the shutter is kept completely covering the orifice, and leakage past it, should be stopped by filling silt against its upstream face. This vent will be regulated by men standing on the plug stone.

In case of large tanks, where the quantity of water to be released is great, and where adequate establishment is maintained, flat shutters working in grooves, and regulated by screw spears, may be installed.

For heads of over 9 m or so, special balanced valves or shutters moving on rollers are generally installed.

Example 26.1. *Design a vertical drop horizontal floor tank surplus weir with the following data. Draw a neat sketch indicating the salient dimensions of the weir and bund section, and work out the length of the weir required with the following given data:*

Combined catchment	*= 26 sq. km.*
Intercepted catchment	*= 20 sq. km.*
Maximum water level	*= + 7.50 (m)*
Full tank level	*= + 6.70*
Ground level	*= + 5.80*
Tank bund level	*= + 1.80*
Slope on either side of bund	*= 2 : 1*
Ryve's coefficient for combined catchment	*= 9.0*
Ryve's coefficient intercepted	*= 1.8*

Solution. Using equation (26.1), we have

Q_P = peak flood discharge

$$= C_1 A^{2/3} - c_1 a^{2/3}$$

where $C_1 = 9.0$, $c_1 = 1.8$

$A = 26$ sq. km., $a = 20$ sq. km.

$$Q = 9.0\,(26)^{2/3} - 1.8\,(20)^{2/3} = 79.2 - 13.3 = 65.9 \text{ cumecs.}$$

or $Q_p = 65.9$ cumecs.

Now, M.W.L. = 7.50 m

F.T.L. = 6.70 m

Ground level = 5.80 m

$\therefore$ Difference between MWL & FTL *i.e.* maximum head of water over surplus weir = $H = 7.50 - 6.70 = 0.8$ m.
crest of weir is kept at F.T.L.

$\therefore$ Depth of drop *i.e.* height of weir crest

$$D = 6.7 - 5.8 = 0.9 \text{ m.}$$

Since the drop is small, we can adopt a masonry weir with a horizontal masonry floor (without depressed floor) *i.e.* type A, Fig. 26.3 (*a*), easily.

The widths of horizontal pucca floor and talus are calculated below :

The width of horizontal masonry floor $= 2(D + H) = 2(0.9 + 0.8) = 3.4$ m.

The width of rough stone talus $= 4(D + H) = 4(0.9 + 0.8) = 6.8$ m.

Use the top crest width equal to 0.6 m, and draw the sketch as shown in Fig. 26.11.

The length of the weir can be determined by using equation (26.2) as below :

$$Q_p = C \cdot LH^{3/2}$$

where $Q_p = 65.9$ cumecs (calculated above)
$L = ?$
$H = 0.8$ m.
$C = 1.84$ for weirs with crests upto 0.9 m wide.

$\therefore$ $$65.9 = 1.84 \times L \times (0.8)^{3/2}$$

or $$L = \frac{65.9}{1.84 \times 0.716} \text{ m} = \mathbf{50 \text{ m.}} \quad \textbf{Ans.}$$

Example 26.2. *A waste weir of a tank has the following details :*

Combined catchment	*= 26 sq. km.*
Intercepted catchment	*= 20 sq. km.*
F.T.L. of tank	*= + 12.70 m.*
Crest level of tank	*= + 12.70*
M.W.L. of tank	*= 13.4*
G.L.	*= 10.00*
Ryve's coefficient for combined catchment	*= 9*
Ryve's coefficient intercepted	*= 1.5.*

Design the length and cross-section of the body wall of the surplus work, assuming the foundation soil to be soft but only slightly pervious.

Solution. Depth of water over weir crest

$$= H = MWL - FTL = 13.4 - 12.7 = 0.7 \text{ metre}$$

The Depth of fall (D) = F.T.L. – G.L. = 12.7 – 10.0 = 2.7 m.

The choice is obviously a vertical drop fall with a water cistern.

Total length of horizontal floor

$$= 3(D + H) = 3(2.7 + 0.7) = 3 \times 3.4 = \mathbf{10.2 \text{ m.}}$$

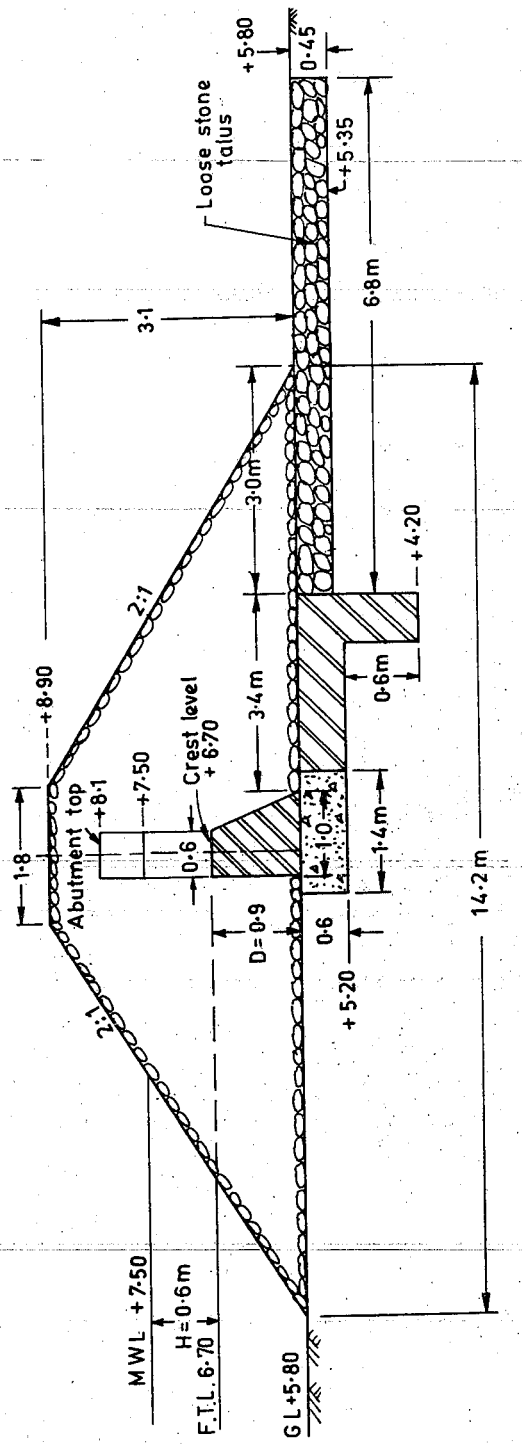


Fig. 26.11. Section of the designed Tank Surplus Weir in example 26.1.

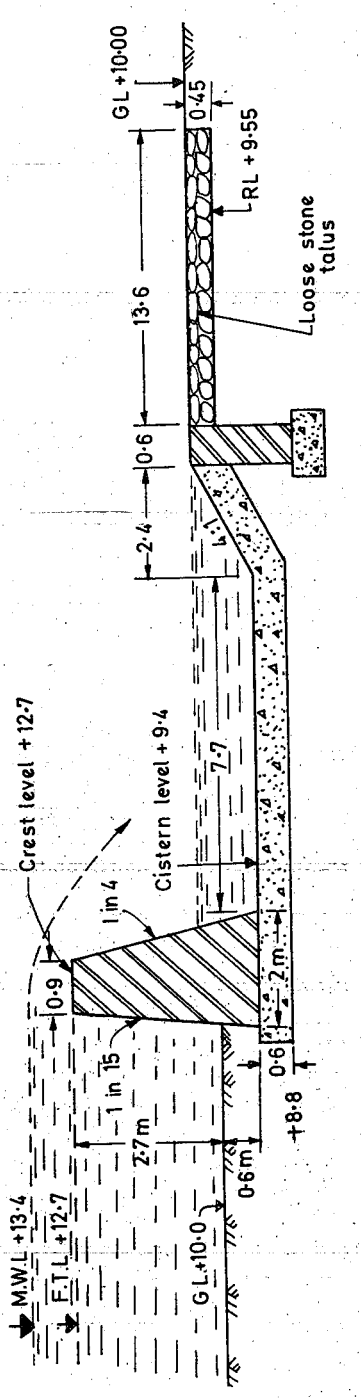


Fig. 26.12. Section of the designed Tank Weir in example 26.2.

Length of d/s talus = $4(D+H) = 4(2.7+0.7) = 4 \times 3.4 =$ **13.6 m.**

The depth to which the horizontal floor is depressed is given by equation (12.9) as $X = \frac{1}{4}(H \cdot H_L)^{2/3}$, where H_L is the difference in u/s and d/s water levels *i.e.* $(H+D)$ in this case.

or $X = \frac{1}{4}(0.7 \times 3.4)^{2/3}$

$= 0.5$; So let us use 0.6 m depth of cistern

Length of cistern is given by equation (12.8) as

$L_C = 5 \cdot \sqrt{H \cdot H_L} = 5 \cdot \sqrt{0.7 \times 3.4} = 7.7$ m.

The slope length of concrete floor in a slope of 4 : 1 will be $= 4 \times 0.6 = 2.4$ m.

d/s cut-off wall thickness $= 0.6$ m

In this way, total length of floor provided

$= 7.7 + 2.4 + 0.6 = 10.7 > 10.1$; safe.

Hence, provide the section of the waste weir, as shown in Fig. 26.12.

Length of weir

$$Q_p = C_1 A^{2/3} - c_1 a^{2/3}$$

where $C_1 = 9.0$

$c_1 = 1.5$

$A = 36$ sq. km.

$a = 20$ sq. km.

$$Q_p = 9(36)^{2/3} - 1.5(20)^{2/3}$$

$$= 9 \times 10.9 - 1.5 \times 7.4 = 98.1 - 11.1 = \mathbf{87\ cumecs}$$

Now $Q_p = CLH^{3/2}$

$87 = 1.84 \times L \cdot (0.7)^{3/2}$

$$\therefore \quad L = \frac{87}{1.84 \times 0.948} = 49.8 \text{ m ; } \mathbf{say\ 50\ m}$$

Hence, use 50 m length of weir. **Ans.**

Example 26.3. *A tank has a catchment area of 120 sq. km. ; out of which 20 sq. km. make free catchment. Average annual rainfall on catchment is 75 cm. What should be the desirable gross storage for the tank ? If rice crop, with a duty of 640 hectares per cumec at the head of main canal and a crop period of 120 days, is to be irrigated from the water of this tank, find the area under the rice crop.*

What should be the length of the waste weir to dispose of safely the high flood discharge, with a head of 0.6 m on the crest of the weir ? Use Ryve's formula, with coefficient as 7.5.

How will the live storage be affected if the tank gets filled $1\frac{1}{2}$ times in a year.

Solution. Average annual rainfall = 75 cm.

The rainfall of average bad year = $\frac{2}{3}$rd of the average annual rainfall(say).

$= \frac{2}{3} \times 75 = 50$ cm.

Runoff (as depth of water over the catchment) of this average bad year

$$= 20\% \text{ (say) of rainfall} = \frac{20}{100} \times 50 = 10 \text{ cm.}$$

The total catchment area of 120 sq. km. consists of independent catchment of 20 sq. km. and intercepted catchment of 100 sq. km.

Total volume of runoff in average bad year from free as well as intercepted catchment of the given tank

= full runoff from the free catchment + $\frac{1}{7}$th of the runoff from the intercepted catchment on the assumption that $\frac{6}{7}$ of the runoff from the intercepted catchment is impounded in the upper tanks.

$$= \left[20\,(1000\times1000)\times\left(\frac{10}{100}\right)+\frac{1}{7}\times100\,(1000\times1000)\,\frac{10}{100}\right]\text{m}^3.$$

$$= \left[\frac{20\times10^6}{10}+\frac{1}{7}\times100\times10^6\times\frac{1}{10}\right]\text{m}^3$$

$$= 10^6\,[2+1.43] = 3.43\times10^6\ \text{m}^3 = 3.43\ \text{million cubic metre.}$$

$\therefore$ Live storage $= 1.1\times3.43 = 3.77\ \text{M.m}^3$

and, Gross storage $= \left(\frac{10}{9}\right)3.77 = \mathbf{4.19\ M.m^3}$ **Ans.**

Water required at the head of the main canal to irrigate 640 hectares of rice crop

$= 1$ cumec for 120 days. $= 1\times(120\times24\times60\times60)$ cu.m.

$= 10.37\times10^6$ cu.m. $= 10.37\ \text{M.m}^3$

Hence, the area of rice crop which can be irrigated from the full tank water

$$= \frac{\text{Effective storage of the tank}}{10.37}\times640\ \text{hectares}$$

$$= \frac{0.9\times\text{Live storage}}{10.37}\times640\ \text{hectares}$$

(assuming the losses in the tank to be 10% of live storage).

$$= \frac{0.9\times3.77}{10.37}\times640 = \mathbf{209.4\ hectares.}\quad \textbf{Ans.}$$

Now, the high flood discharge from the catchment into this tank (after the tank is full) is given by equation (26.2) as

$$Q_p = C_1A^{2/3} - c_1a^{2/3},$$

where $C_1 = 7.5$ (given)

$c_1 = \frac{1}{5}C_1$ (assumed) $= 1.5$

A = Full combined catchment = 120 sp. km. (given)

a = Intercepted catchment = 100 sq. km. (given)

$\therefore \quad Q_p = 7.5\,(120)^{2/3} - 1.5\,(100)^{2/3} = 183 - 32.4 = 150.6$ cumecs.

Now, if L is the length of the waste weir, then

$$Q_p = C\cdot L\cdot H^{3/2} = 1.84\cdot L\cdot H^{3/2}$$

$\therefore \quad 150.6 = 1.84 \times L \times (0.6)^{3/2}$

or $L = \mathbf{176\ m.}$ **Ans.**

If the tank fills $1\frac{1}{2}$ times in a year, then the live storage

$$= \frac{3.77}{1.5} = \mathbf{2.51\ M.m^3} \quad \textbf{Ans.}$$

PROBLEMS

1. What is an irrigation tanks ? Differentiate between isolated tanks and group of tanks. how can your compute the storage capacity of an irrigation tank ?

2. (*a*) What are the causes of failure of tank bunds ?

[**Hint.** Refer article 20.7]

(*b*) Sketch a typical cross-section of a tank bund of type C. Under what circumstances such sections are chosen ?

(*c*) Explain briefly the function of a breaching section in a tank bund.

3. Design a tank surplus weir with the following data. Draw a neat sketch (not to scale) indicating the salient dimensions.

Combined catchment	= 26 sq. km
Intercepted catchment	= 20 sq. km.
Full tank level	= + 6.70 (m)
Maximum water level	= + 7.50 m
Ground level	= + 5.80 m
Foundation level	= + 4.20 m
Tank bund level	= + 8.90 m
Top width of bund	= 1.8 m
Slope on either side of bund	= 2 : 1

Downstream level is + 4.80 in a distance of 8 m.

4. A tank surplus work has the following details :

Combined catchment	= 26 sq. km.
Intercepted catchment	= 20 sq. km.
F.T.L. of tank	= 12.00 m.
Crest level of weir	= 12.00 m.
M.W.L. of tank	= 12.75 m.
G.L.	= 11.00 m.
Top of foundation concrete	= 9.60 m
Ryve's coefficient for combined catchment	= 9
Ryve's coefficient intercepted	= 1.5

Design the length and cross-section of the body wall of the surplus work.

5. What is a waste weir, and where is it installed in a project of an irrigation tank ? Discuss its importance, and principles involved in designing such a work.

6. (*a*) What is a tank weir, and what are its types ? Discuss the conditions favourable to the selection of a particular type.

(*b*) How does the foundation and working conditions differ for a tank weir as against a river weir.

(*c*) Where should a tank weir be located in a tank irrigation project ?

7. (*a*) What are dam-stones and where are they installed ? What are their advantages and disadvantages ?

8. What is a surplus weir and its types in an irrigation tank project ? Draw net sketches of the sections of A type weirs with and without water cushion provision.

9. (*a*) What do you mean by a tank sluice, and where is it installed ?

(*b*) Describe the different types of tank sluices indicating the regulation arrangements.

(*c*) For the data given below, design and sketch a tank sluice :

Discharge	= 0.25 cumec
Sill level of sluice	= + 20.00 m.
Top of bank	= + 56.00 m.
Assumed head of discharge	= 0.3 m.
Side slopes of banks in front and rear	$= 1\frac{1}{2} : 1$

Assume any other data, if required.

10. Write short notes on :

(*i*) Free catchment and combined catchment (*ii*) Isolated tanks and tanks in series :

(*iii*) Tank bunds (*iv*) Tank sluices ; and

(*v*) Tank weirs.

27

Arch and Buttress Dams

ARCH DAMS

27.1. Definition and Types of Arch Dams

An arch dam may be defined as a solid wall, *curved in plan*, standing across the entire width of the river valley, in a single span*. This dam body is usually made of cement concrete, although rubble and stone masonry has also been used in the past.

This wall will structurally behave : partly as a *cantilever retaining wall standing up from its base, and partly, the load will be transferred to the two ends of the arch span by horizontal arch action.* The arch load will, thus, be transferred to the side walls of the canyon, which must be strong, stable and rocky.

The distribution of part of the load to the side walls of the canyon, reduces the load on the cantilever wall, thereby reducing its thickness, as compared to that in an ordinary gravity dam ; and that is the only benefit we derive from an arch dam in comparison to a gravity dam.

Evidently, the greater is the wall curvature (in plan), the greater will be the load that will be transferred to the sides of the canyon, and hence greater will be the economy in the dam thickness.

This economy in dam thickness can be further increased considerably by making the dam body not only curved in plan, but also curved in section. Such a non vertical dam is known as **double curvature arch dam** or a **shell-arch dam**, because such dams are designed as *shell-structures*. Such three dimensional designs are quite complex, and are taught only at M-Tech level in Structural Engineering.

Since the design and construction of an arch dam is very complicate, requiring extraordinary skill for erecting shuttering in the field, it is generally preferred in practical life to construct gravity dams. And that is why, we find only one arch dam in our country**, as against several hundreds of gravity dams. This arch dam too, is not a simple arch dam, but a shell-arch dam.

Simple arch dams, which transfer a large part of their loading by cantilever action, may also be of different types, since their faces may be either vertical or curvilinear. Depending upon the shape consideration, simple arch dams can be divided into three types, *viz :*

(*i*) *Constant radius arch dams*

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* When multiple or a number of arches are used, supported between intermediate piers, the dam is known as a *buttress dam.*

** *Idukki dam*, across Periyar river in Kerala State ; Photoview shown in Fig. 27.1.

(*ii*) *Variable radius arch dams ;* and

(*iii*) *Constant angle arch dams.*

A constant radius arch dam is the simplest in design as well as construction, but uses the maximum concrete. A constant angle arch dam on the other hand, uses about 43% of the concrete used by a *constant radius arch dam.* The variable radius arch dam is an intermediate choice, using around 58% of the concrete used by *constant radius arch dam.*

The *shell arch dams* are much more economical than even the constant angle arch dams, as their sections can be quite thin. Say for example, the famous *Vajont dam* of Italy is only 22 m thick at its base, inspite of being 261.6 m in height. Similarly, the *ldduki dam* in India is only 45 m thick at its base, even with 170.7 m height. As compared to these thin sections, the famous *Hoover dam* of USA, which is a *constant radius arch dam (with upstream face vertical) is 201 m thick at its base, with only 222 m height. The difference between the three types of arch dams are explained below :*

27.1.1. Constant Radius Arch Dams. (Fig. 27.2). A constant radius arch dam is that, in which, the radii of the *outside** curved surface are equal at all elevations, from top to the bottom. The centres of all such circular arcs, called **extrodos**, will therefore, evidently lie on one vertical line. However, the **introdos** (*i.e.* inside** curved surface of the arch) has gradually decreasing radius from top to the bottom, so as provide increased concrete thickness towards the base for accounting the proportionally increasing hydrostatic water pressure of the reservoir. The dam body will, therefore, be triangular in cross-section with upstream face vertical, and a minimum thickness at the top.

Evidently, it is only the radii of the *introdos*, which decrease with depth ; while the centres of all such circular arcs continue to lie on the same vertical line, on which lie the centres of the *extrodos.* Hence, in such a dam, the centres of extrodos, introdos, as well as the centrelines of the horizontal arch rings. at various elevations, lie on a straight vertical line that passes through the centre of the horizontal arch ring at the crest. Such a dam is, therefore, sometimes called a **constant centre arch dam**, although strictly speaking, this centre is not at one point, but lies at different heights along one vertical line.

Evidently, the central angles of the arch rings of the introdos will vary at different elevations, due to the varying width of the river valley (see Fig. 27.2); the maximum being at the top of the dam, and the minimum at the bottom of the dam.

It has further been shown that the *best* or *most economical central angle* in an arch dam is the one whose value is equal to 133° – 34'***. But in a constant radius arch dam, such an angle value can be adopted only at one place, since the angle varies with height considerably, due to narrow V-shape of the valley. It is therefore considered prudent to keep the economical angle of 133° – 34' at about mid height. The angle at the top will, therefore, be more than this value; but due to topographical considerations, the angle at the top can not be fixed greater than about 150°. *Hence, the angle at the top should be such as to give the best average angle, but restricted to about 150°.*

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* Water side or upstream side.

** Downstream side.

*** This is derived in article 27.3.1.1, a little later.

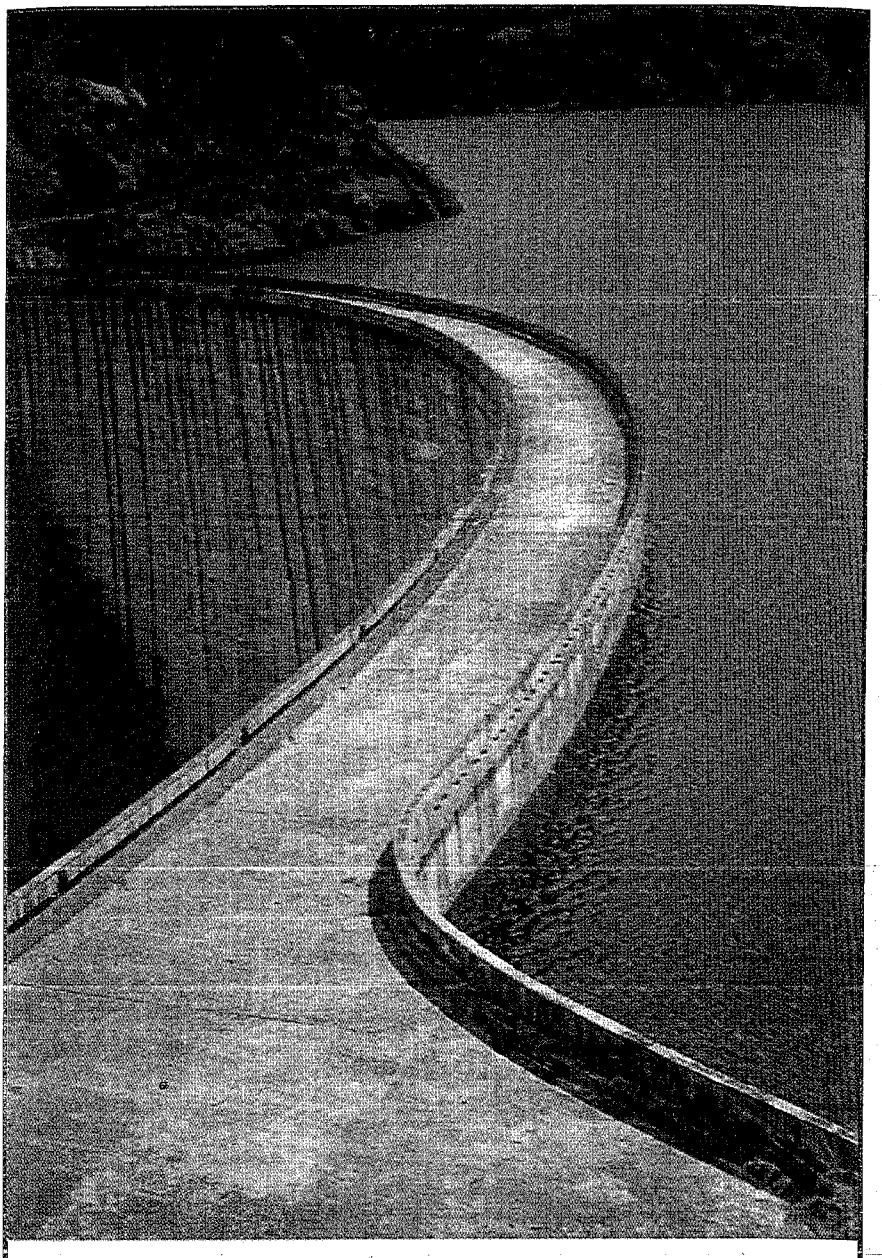


Fig. 27.1. Photoview of Idukki Dam.
A concrete arch dam, constructed across Pariyar river in Kerala State of India. The length of the dam at top is 366 m and maximum height above the lowest point of foundation is 169 m. The construction of the dam was completed during 1966-1974.

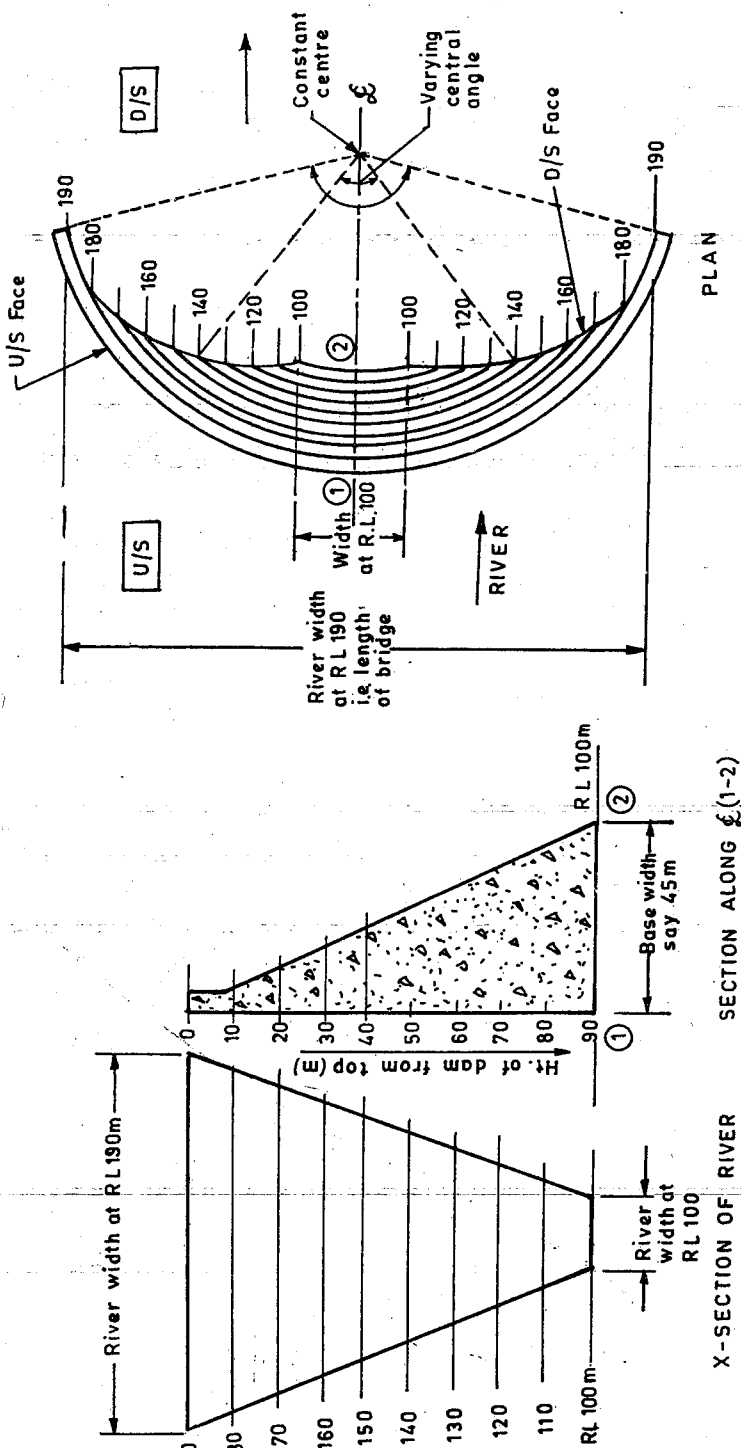


Fig. 27.2. Constant radius arch dam.

27.1.2. Variable Radius Arch Dams. (Fig. 27.3). A variable radius arch dam is the one in which the radii of the extrodos curves* and of *introdos curves*** vary at various

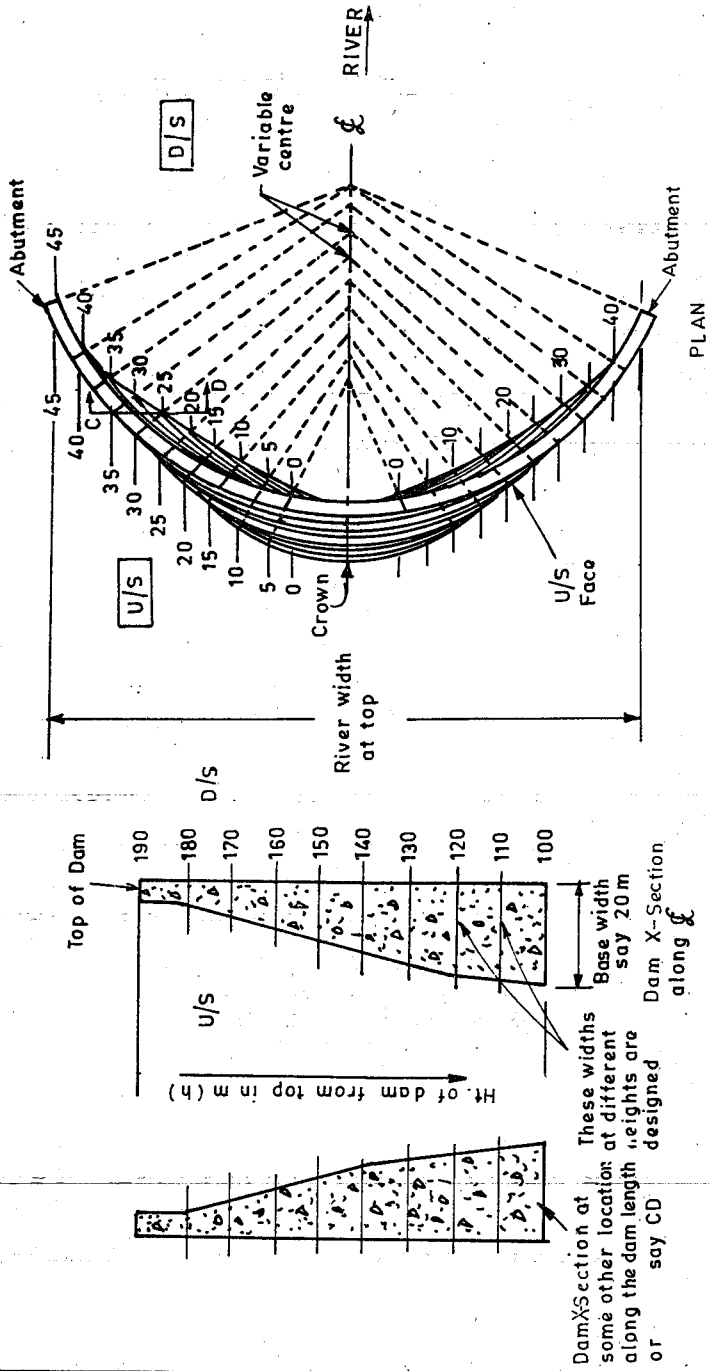


Fig. 27.3. Variable radius arch dam.

* arch rings corresponding to upsteam face.

** arch rings corresponding to downstream face.

elevations, being maximum at the top, and a certain minimum at its bottom. This makes the central angles as large as possible, so that the maximum arch efficiency may be obtained at all elevations.

In a typical design of such a dam, the downstream face of the dam at the central line (crown) is vertical ; while at all other locations, there is a batter on both the sides except at the abutments, where again, the upstream side becomes vertical. If overhangs are permitted, due to availability of stronger foundations, then the faces at the crown as well as abutments, may be provided with overhangs, affecting saving in the designed thicknesses.

Evidently, since in such an arch dam, the centres of the various arch rings at different elevations, do not lie on the same vertical line, it is also known as **variable centre arch dam**. Such dams are preferred for V-shaped valleys as compared to constant radius arch dams which may be preferred for comparatively wider U-shaped valleys.

27.1.3. Constant Angle Arch Dams. The constant angle arch dam is a special type of variable radius arch dam, in which the central angles of the horizontal arch rings are of the same magnitude at all elevations, as shown in Fig. 27.4. The design of such a dam can, thus, be made by adopting the best central angle of 133° – 34′ ; and hence such a dam proves to be the most economical, out of the three types of ordinary arch dams, as pointed out earlier also.

However, the design of such a dam usually involves providing *overhangs at abutments*, which require stronger foundations, and hence such a type cannot be used if the foundations are weak.

27.2. Forces Acting on Arch Dams

Generally, the same forces act on an arch dam, which do act on a gravity dam. These forces are : (*i*) *water pressure* ; (*ii*) *uplift pressure* ; (*iii*) *earthquake forces* ; (*iv*) *silt pressure* ; (*v*) *wave pressure* (*vi*) *ice pressure* ; as discussed in article 19.3. However, the relative importance of the forces is different in an arch dam, as compared to that in a gravity dam. Say for example, the uplift pressure in an arch dam is small and is generally neglected, because of the narrow base width of its body. On the other hand, the stresses caused by *ice, temperature changes*, and *yield of supports (i.e. abutments)*, generally become quite important in arch dams, and hence must be thoroughly examined.

Whereas, the ice pressure, applicable in cold countries, causes a continuous concentrated load along the arch element at the elevation of the ice ; the internal stresses caused by the temperature changes move the dam upstream during the summer, and downstream during the winter. Hence, the **low water temperatures** become quite important in stress analysis, since these stresses act additive to the reservoir water pressure. Moreover, even the slight **yield of abutments** due to transfer of load by arch action, may also cause high internal stresses in the arch, and should therefore, be prudently accounted for.

27.3. Design of Arch Dams

Arch dams can be designed on the basis of any one of the following three methods:

(1) *Thin cylinder theory* ;

(2) *Theory of Elastic arches ; and*

(3) *The Trial load method.*

radius r, angle subtended at the centre of the arch equal to 2α, and thickness t. Intensity of *hydrostatic pressure* p at any depth h, is given by :

$$p = \gamma_w h \qquad ...(i)$$

where γ_w = unit weight of water.

This horizontal pressure acts in the radial direction. The total downstream component of the horizontal force (P_H), acting along the axis of the river, is then given by

$$P_H = \text{Intensity of pressure} \times \text{Projected area} = \gamma_w \cdot h \times 2r \sin \alpha$$

$$= 2\gamma_w h r \sin \alpha \qquad ...(ii)$$

If R is the reaction at each abutment, its upward component is equal to $2R \sin \alpha$. Equating this to P_H for equilibrium, we get

$$2\gamma_w \cdot h \cdot r \sin \alpha = 2R \sin \alpha$$

or $$\gamma_w \cdot h\, r = R \qquad ...(iii)$$

This equation gives an expression for the abutment reaction, which is equal to the maximum compressive force induced in the arch.

If σ is the compressive stress induced in the arch at the abutments, then

$$\sigma = \frac{R}{t} = \frac{\gamma_w \cdot h \cdot r}{t} \qquad ...(iv)$$

If f_c is the allowable compressive stress for the arch material, then

$$f_c \gg \frac{\gamma_w \cdot h \cdot r}{t}; \text{ or } t \gg \frac{\gamma_w \cdot h \cdot r}{f_c}$$

Under Limits, $$\boxed{t = \frac{\gamma_w \cdot h \cdot r}{f_c}} \qquad ...(27.1)$$

The above equation indicates that the thickness of the arch (t) should increase linearly with the depth below the water surface, and that at a given elevation, the required thickness of the arch ring is proportional to the radius of the arch (r) at that elevation.

27.3.1.1. Central angle for minimum concrete. Considering the arch ring as shown in Fig. 27.5, we have

the volume of concrete per unit height of the arch ring (V)

$$= r\,(2\alpha) \cdot A$$

where A is the cross-sectional area of the arch ring $= (t \times 1)$

Since $A = t \times 1$, we have

$$V = r\,(2\alpha) \cdot t$$

Substituting the value of t from Eq. (27.1), we get

$$V = r \cdot (2\alpha) \cdot \frac{\gamma_w \cdot h \cdot r}{f_c}$$

or $$V = \frac{\gamma_w \cdot h}{f_c} \cdot r^2 \cdot (2\alpha) \qquad ...(i)$$

Now, if L is the span of the arch ring, then from Fig. 27.5, we observe that

$$\sin \alpha = \frac{L}{2r}$$

or $$r = \frac{L}{2 \sin \alpha} \quad ...(ii)$$

Substituting this in Eq. (*i*) above, we get

$$V = \frac{\gamma_w \cdot h}{f_c} \left(\frac{L}{2 \sin \alpha} \right)^2 (2\alpha)$$

$$= \frac{\gamma_w \cdot h}{f_c} \cdot \frac{L^2}{2} \left[\frac{\alpha}{\sin^2 \alpha} \right] \quad ...(iii)$$

where $\frac{\gamma_w \cdot h}{f_c} \cdot \frac{L^2}{2}$ is a constant $= K$ (say)

$\therefore$ $$V = K \cdot \left[\frac{\alpha}{\sin^2 \alpha} \right]$$

or $$\frac{\partial V}{\partial \alpha} = K \left[\frac{\sin^2 \alpha - 2\alpha \cdot \sin \alpha \cos \alpha}{\sin^4 \alpha} \right]$$

For minimum volume of concrete (V)

$$\frac{\partial V}{\partial \alpha} = 0$$

$\therefore$ $$\sin^2 \alpha = 2\alpha \cdot \sin \alpha \cdot \cos \alpha$$

or $$\sin \alpha = 2\alpha \cdot \cos \alpha$$

or $$\frac{\sin \alpha}{\cos \alpha} = 2\alpha$$

or $$\tan \alpha = 2\alpha$$

or $$2\alpha = \tan \alpha$$

$\therefore$ $$2\alpha = 133° - 34'$$

This shows that the most economical central angle is $133° - 34'$, as was stated by us earlier.

27.3.1.2. Limitations of thin cylinder theory. The various limitations of this theory are :

(*i*) The arch sections are not thin cylinders. They are also not free at abutments, as assumed in this theory.

(*ii*) The theory does not consider shear and bending stresses in the arch.

(*iii*) The analysis is based on only the hydrostatic water pressure. *Temperature stresses and ice pressures, which are quite important in arch dams, get ignored in this theory.*

(*iv*) Stresses due to yielding of abutments and those due to rib shortening have not been accounted in this theory.

(*v*) plastic flow of concrete and shrinkage in concrete have not been accounted for.

27.3.2. The Elastic Theory for Design of Arch Dams. As pointed out earlier, the archs are subjected to additional forces, such as :

(*i*) *temperature stresses*, due to temperature changes ;

(*ii*) *shrinkage stresses*, due to setting of concrete ;

(*iii*) stresses due to yielding of abutments ; and

(*iv*) stresses due to rib shortening.

The stresses at (*i*) and (*ii*) are quite evident, and need no explanation. It may however, be added that the effect of rise in temperature, as in summer season, produces opposite stresses, whereas, the stresses due to fall in temperature are additive, and hence, only need be accounted. The stress at (*iii*) above is also understood, since no abutment is rigid, and yields to an extent depending upon the nature of the rock. This yield increases the span of the arch, inducing additional stresses. The stress at (*iv*) above, can be explained as below :

In case of a cylinder, the pressure acts from all sides, and the entire ring is compressed so that the diameter of the cylinder decreases by a small amount, but *there* is no deformation. While in the case of an arch, *which is a part of a cylinder*, the ends being fixed, the distance between them (*i.e.* the span) does not change. This restriction of the change in span, causes additional stresses, known as *rib- shortening stresses. In a long thin arch, these stresses are small ; but in the case of a thick arch of small angle, they are appreciable, and need to be accounted for.*

All the above stresses are taken into consideration by the theory of elastic arches.

27.3.3. General formulas. The formulas for elastic arches, usually given in the books on "Theory of Structures" or "Applied Mechanics" are reproduced here for the benefit of students, intending to solve arch dams by elastic theory.

With reference to Fig. 27.7.

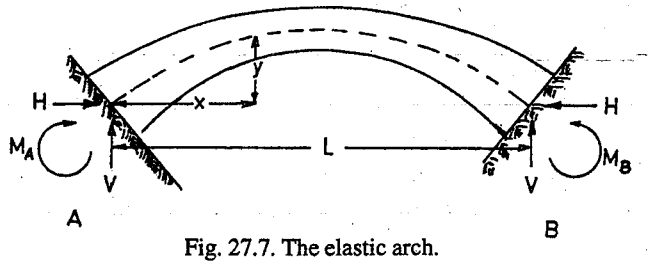


Fig. 27.7. The elastic arch.

$$M = \mu + M_A + (M_B - M_A)\frac{x}{L} + H \cdot y \qquad ...(27.2)$$

$$\int \frac{M}{E \cdot I} y ds = \int \frac{\mu \cdot y}{EI} \cdot ds + M_A \int \frac{y \cdot ds}{EI} + \frac{M_B - M_A}{L} \int \frac{xyds}{EI} + H \int \frac{y^2}{EI} ds = 0 \qquad ...(27.3)$$

$$\int \frac{Mx}{EI} ds = \int \frac{\mu \cdot x}{EI} \cdot ds + M_A \int \frac{x \cdot ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x^2 \cdot ds}{EI} + H \int \frac{xy}{EI} ds = 0 \qquad ...(27.4)$$

and $$\int \frac{M}{E \cdot I} ds = \int \frac{\mu}{EI} \cdot ds + M_A \int \frac{ds}{EI} + \frac{M_B - M_A}{L} \int \frac{x \cdot ds}{EI} + H \int \frac{yds}{EI} = 0 \qquad ...(27.5)$$

where μ = the bending moment on a straight horizontal freely supported beam

M = the bending moment at any point on the arch

M_A and M_b = fixed end bending moments at both the ends of the arch

H = horizontal thrust at the abutments

For **symmetrical arch**, $M_A = M_B$; and the Eq. (27.4) vanishes ; while Eqs. (27.3) and (27.5) get simplified to :

$$\int \frac{M}{EI} \cdot y \cdot ds = \int \frac{\mu \cdot y}{EI} \cdot ds + M_A \int \frac{y ds}{EI} + H \int \frac{y^2 \cdot ds}{EI} = 0 \qquad ...[27.3\ (a)]$$

and $$\int \frac{M}{EI} \cdot ds = \int \frac{\mu}{EI} ds + M_A \int \frac{ds}{EI} + H \int \frac{y \cdot ds}{EI} = 0 \qquad ...[27.5\ (a)]$$

As the *contraction* due to temperature effect and *rib- shortening* due to thrust, is prevented by the abutments, these effects modify the above Eqs. (27.3), (27.4) and (27.5) as

$$\alpha \cdot TL - \int \frac{M}{EI} \cdot y \cdot ds = 0 \qquad ...[27.3\ (b)]$$

where α = Coefficient of thermal expansion

T = Fall in temperature in °C

$$\int \frac{M}{EI} ds = 0 \qquad ...[27.4\ (b)]$$

$$\int \frac{M \cdot x}{EI} ds = 0 \qquad ...27.5\ (b)]$$

and $$M = M_A - V_x + H \cdot y \qquad ...(27.6)$$

in which the additional term V stands for the vertical thrust of opposite nature at the two abutments, and is reckoned positive upwards.

For a symmetrical arch, $V = 0$; so that the Eq. (27.6) simplifies to

$$M = M_A + H \cdot y \qquad ...[27.6\ (a)]$$

and Eqs. [27.4 (*b*)] and [27.5 (*b*)] become

$$M_A \int \frac{y}{EI} \cdot ds + H \cdot \int \frac{y^2}{EI} ds - L \cdot \alpha T = 0 \qquad ...[27.4\ (c)]$$

and $$M_A \int \frac{ds}{EI} + H \int \frac{y}{EI} ds = 0 \qquad ...[27.5\ (c)]$$

By solving these equations, the values of M_A, H and V are obtained.

Rib-shortening effect can be reckoned along with the other loads by adding its effect to the left side of Eq. (27.3).

BUTTRESS DAMS

27.4. Definition and Types of Buttress Dams

An ordinary concrete gravity dam, as we know, is a solid body of mass concrete, somewhat triangular in section, running across the entire width of the river valley. Such a solid wall requires huge amount of concrete, which partly remains *unstressed to full extent*, thereby leading to wastage of concrete. The uplift pressure also acts on the entire width of the dam body, from its bottom, which further increases its size, without giving any additional benefit as a dam.

Efforts have, therefore, been made from time to time, to innovate methods for affecting economy in the use of concrete, by cutting down the concrete from those dam portions where it remains unstressed. Attempts have, therefore been made to provide *hollow gravity dams*. **Buttress dams**, are an improvement innovation over the hollow concrete gravity dams.

In such an innovated dam, therefore, *solid walls of specified thickness and section*, are constructed parallel to the flow, at some suitable intervals. These walls are called **buttresses.** *Inclined slabs* or *arch slabs* are then supported on upstream side on these buttresses, as shown in Fig. 27.8.

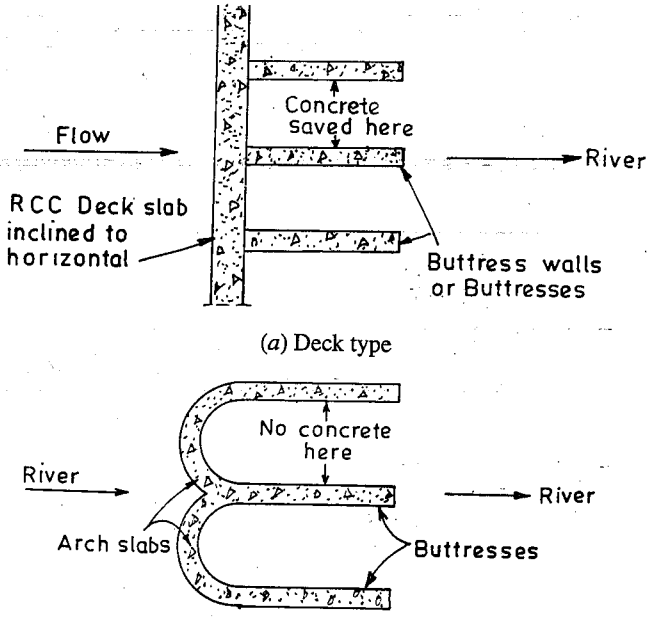


Fig. 27.8. Simplified line sketches of buttress dams.

Although, several types of buttress dams have been devised, but the most common types, are :

1. *Deck slab type ; and*
2. *Multiple arch type.*

Both these types of dams have been shown in their simplified line diagrams, in Figs. 27.7 (*a*) and (*b*) respectively.

Slab type dams are generally preferred for smaller heights, say from 20 to 50 m, or so. The highest such dam in the world is *Rodriguez dam* in Mexico, having a height of 73.2 m. Such dams are virtually referred to as buttress dams, or **Amberson dams** (based upon the name of their inventor).

The multiple arch dams, on the other hand, are used for higher dam heights, say from 50 m onward, and the highest such dam of the world is *Manicougan-5 dam* in Canada, having a height of 210 m.

Detailed views of both these types of dams are shown in Figs. 27.9 and 27.11, respectively ; and are discussed below :

27.4.1. Simply Supported Slab Type of Buttress Dams. (Fig. 27.9). In such a type of dam, the R.C.C. deck slab is freely supported on the buttresses, which have corbels for the slab to rest on. The deck slab is inclined to horizontal by about 40° to 55°, so as to support the dead load of a portion of reservoir water, and thus, to provide the stabilising force, in addition to the 'self' weight of the dam, to avoid its sliding.

Design considerations. The loading and safety criteria for buttress dams or buttresses, is the same as that for a gravity dam section, except that the provided *buttress thickness* '*t*' will take the load coming from the dam length = $x + t$; where x is the clear

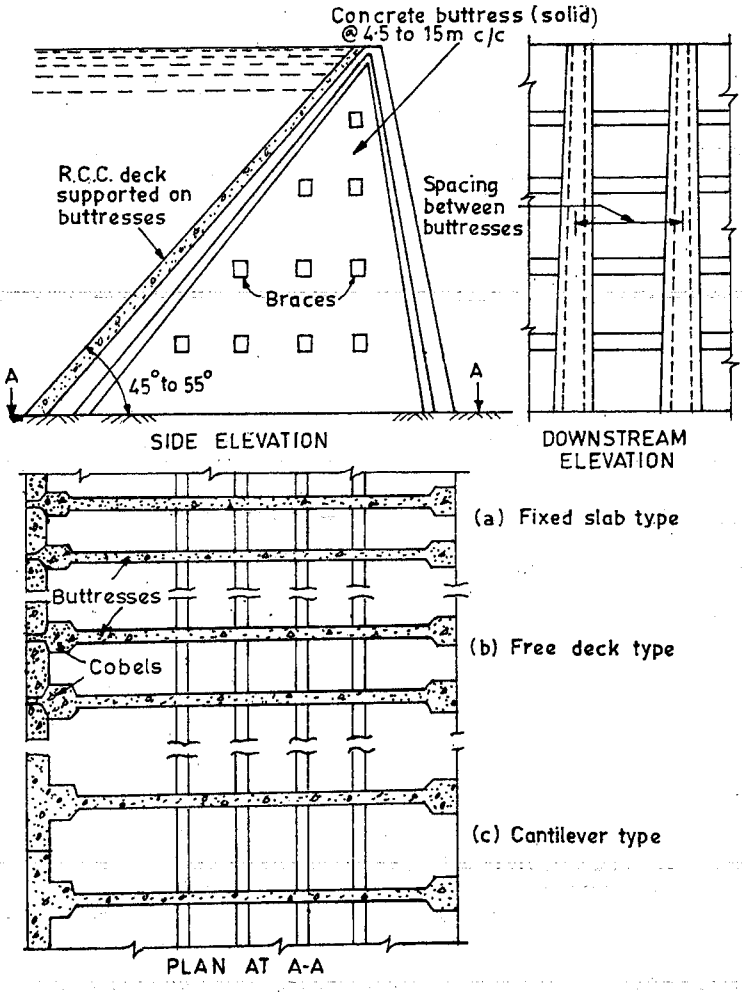


Fig. 27.9. Detailed views of a free deck type of a buttress dam.

spacing between the two consecutive buttresses. Hence, t metre length of buttress section, will take the loads coming from $(x + t)$ metre length of dam, as against the unit metre length of gravity dam section taking load from the unit metre dam length. The resultant impact can be considered by increasing the unit weight of water by multiplying its actual value by a *surcharge factor* (S), defined as :

$$S = \frac{x + t}{t} \qquad ...(27.7)$$

Hence, the effective unit wight of water can be considered as $\gamma_w \left(\frac{x+t}{t}\right)$, and the section of the buttresses can be designed, exactly in the same manner, as a gravity dam section is designed, considering unit length and a continuous section.

The deck slabs can be designed as simply supported R.C.C. decks, each spanning over two adjacent buttresses, and each having a span $= x + t$.

Buttress spacing and Deck slope

(*i*) The **most economical spacing of buttresses** is the one, in which *the minimum thickness of concrete is fully utilised.* This spacing is governed to a very large extent by the values of the upstream slope of the dam (ϕ). For economy, it is also necessary that the buttress spacing be changed for dams of different heights. The suggested spacings for different dam heights, are shown in table 27.1.

Table 27.1. Suggested Economic Buttress Spacings

Mean Dam height in m	*Economic buttress spacing suggested in m,, for a normal ϕ value of 40° to 50°*
Less than 15	4.5
15—30	4.5 to 7.5
30—45	7.5 to 12
above 45	12 to 15

(*ii*) The *height, thickness*, and *spacing of buttresses*, can further be controlled by :

$$\textbf{Slenderness ratio} = \frac{\text{Height of buttress}}{\text{Thickness of buttress}} = 12 \text{ to } 15 \qquad ...(27.8)$$

$$\textbf{Massiveness factor} = \frac{\text{Spacing of buttresses}}{\text{Thickness of buttress}} = 2.5 \text{ to } 3 \qquad ...(27.9)$$

(*iii*) For constant buttress spacing, variation of few degrees in the u/s slope may result in an appreciable change in the quantity of concrete. This concrete quantity required per metre length of dam (V_c) is, infact, related to *u/s* deck slope (ϕ), dam height (H), and the sliding factor $\left(F_s \; i.e. \; \frac{\Sigma H}{V}\right)$ by the equation :

$$V_c = 0.208 \, H^2 \left[\frac{1}{F_s} - \cot \phi\right] \qquad ...(27.10)$$

This equation clearly shows that for a fixed dam height (H) and a given value of F_s, the concrete quantity depends solely on ϕ. Hence, for different values of ϕ, at fixed values of H and F_s, concrete quantities can be worked out. *Vertical lines*, like those shown in Fig. 27.10, are, thus, drawn for each value of the concrete quantity corresponding to each value of ϕ.

Concrete quantities are also worked out for various angles of u/s slope (ϕ) and different buttress spacings, for fixed decided dam height (H) ; and plotted to get a curve for each angle. A **master curve** is then drawn through the junctions of the curves and the vertical lines corresponding to each angle.

The master curve gives the **absolute economic value of buttress spacing** for different *u/s* slope (ϕ) for the decided dam eight.

Say for example, from the plotted curves of Fig. 27.10, we find that for a $\phi = 48°$, buttress spacing is about 8.8 m ; for a $\phi = 49°$, buttress spacing is about 10 m. *This reflects that by increasing the upstream slope, we can increase the buttress spacing.* This figure also reflects that lower values of ϕ (lesser than 47°) are not available on this particular master curve ; which shows that such lower values will not prove economical

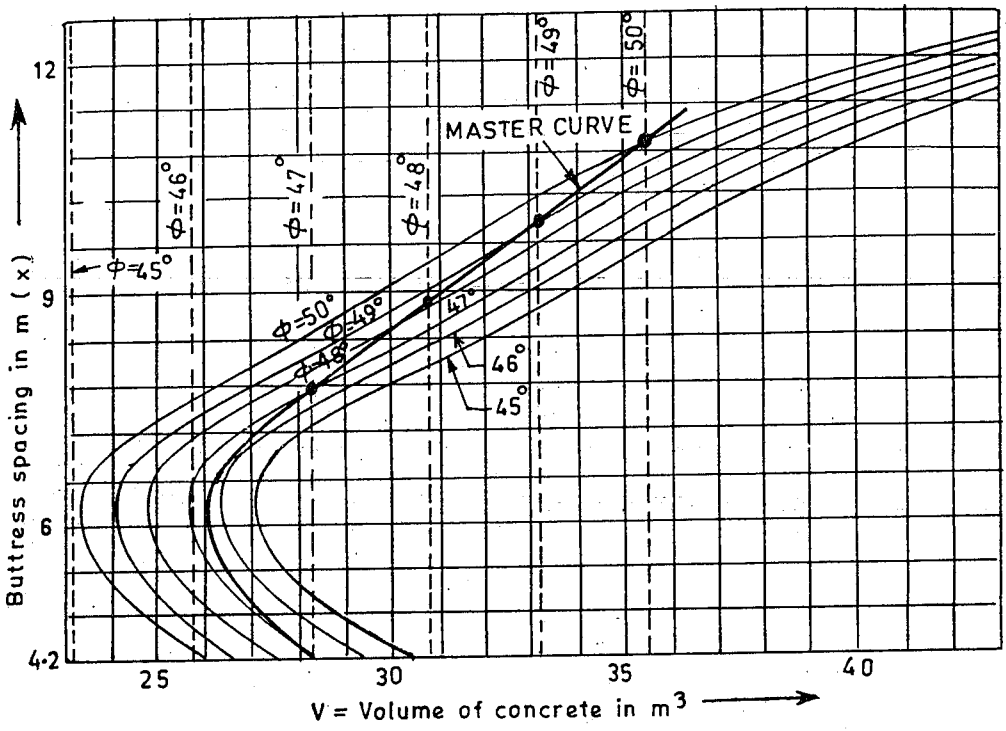


Fig. 27.10. Master curve for absolute economic value of buttress spacing.

for this project. Hence, each individual project (or dam) has, thus, to be analysed for decided dam height, to determine the permissible economical values of ϕ, and their corresponding values of buttress spacing, and thus finally to choose any one set of them.

27.4.2. Multiple Arch Type of Buttres Dams. As stated earlier, in this type of dams, arch slabs are constructed for *u/s* face of the dam, to be supported on buttresses, as shown in Fig. 27.11 and 27.12. Each buttress may be in the form of a single stiffened wall, as shown in Fig. 27.13 (*a*) ; or a double hollow wall, as shown in Fig. 27.13 (*b*).

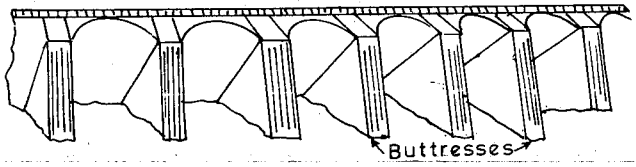


Fig. 27.11. Perspective view of a multi-arch buttress dam.

Meer Alam dam in India, built in about 1800 A.D., is the earliest recorded example of such a type of dam. As stated earlier, such dams are preferred for larger heights, as they prove more stable and flexible, as compared to the ordinary buttress dams (*i.e.* slab type).

The stability of any unit of a multiple arch dam depends on the adjacent units, so that any settlement in one would affect the others. Hence, such a dam would require

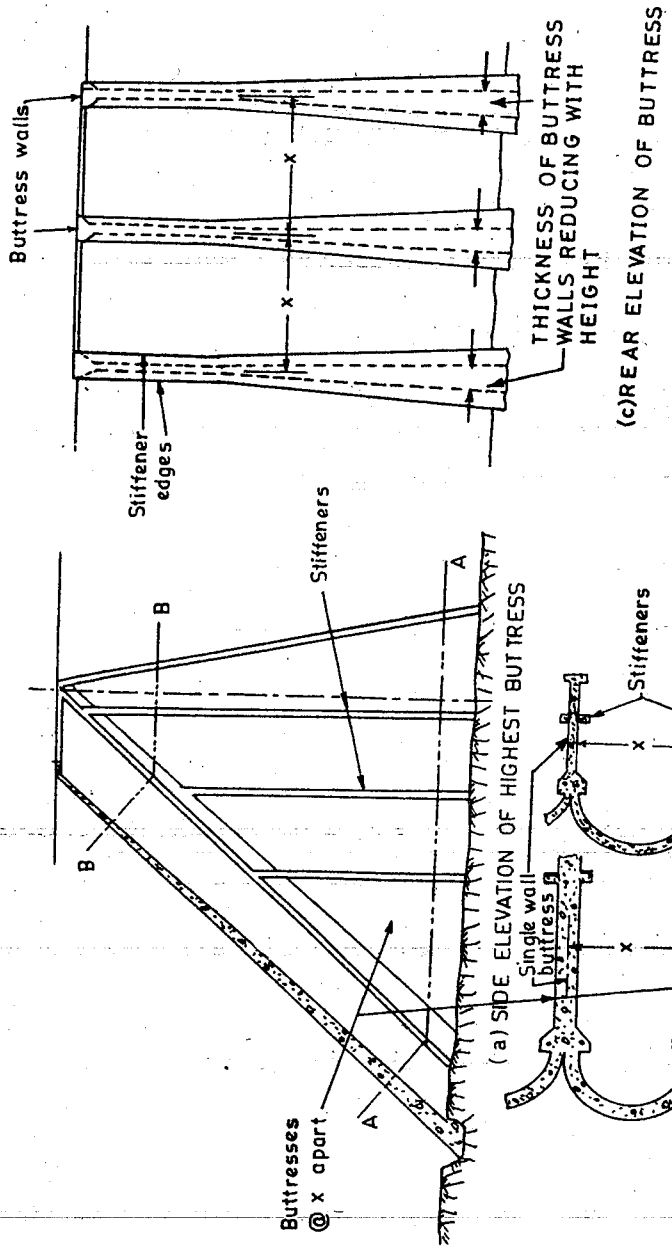
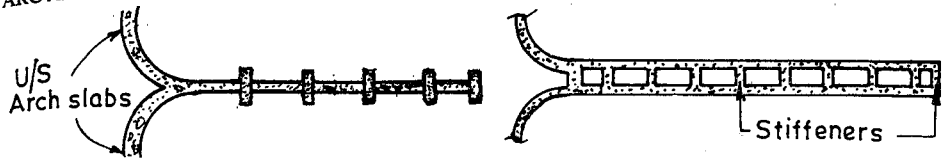


Fig. 27.12. Detailed views of a multiple arch dam.



(*a*) Single wall buttress (*b*) Double wall or hollow buttress

Fig. 27.13. Single and Double walled buttress.

better foundations, as compared to an ordinary buttress dam, where deck slabs on two adjacent buttresses, behave as independent units.

For short spans (10 to 15 m), circular arches of uniform thickness prove economical; whereas, for larger spans, arches of variable thickness are provided. If the central angle of each arch is kept between 180° to 150°, with semi circular or nearly semi circular arches, the horizontal thrust between adjoining arches gets eliminated.

For arched buttress dams, the range of buttress spacing generally adopted, is between 15 to 21 m, although as high a value as 42 m has been permitted in *Webber Greek dam* in California, USA ; a site plan of which is reflected in Fig. 27.14.

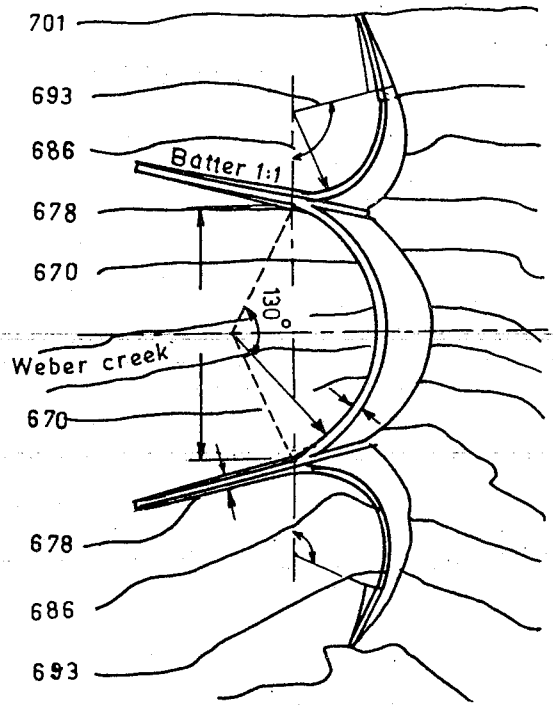


Fig. 27.14. Site plan of Webber creek dam (an example of long span-multiple arch dam).

27.4.3. Other Types of Buttress Dams. In addition to the above described two general types of buttress dams, certain other types, such as *multiple dome type ; massive head type ; columnar buttress type ;* etc., have also been adopted at some places, as discussed below :

27.4.3.1. Multiple dome type. Such a type of buttress dam is quite understandable, as it uses a number of *domes* instead of a number of arches as used in a multiple arch type of dam, and all other features remaining the same. Such a dam can help in

increasing the buttress spacing, but requires sites, where stable foundations can be had at greater depths. The *Coolidge dam on San Carlos river in Arizona is an example of this type of a dam, which is 76.2 m high with buttresses @ 54.8 m apart.*

27.4.3.2. Massive head types. This type of buttress dam is the one, which does not use slabs or archs for upstream face, but the heads of the buttresses themselves are enlarged to meet each other, and thus, to form a continuous water supporting surface, as shown in Fig. 27.15. When the enlarged heads are *cylindrical* on the outerside, it is known as *round head deck* (Fig. 27.15 *a*) ; and when they converge to a point, it is known as a *diamond head deck* (Fig. 27.15 *b*).

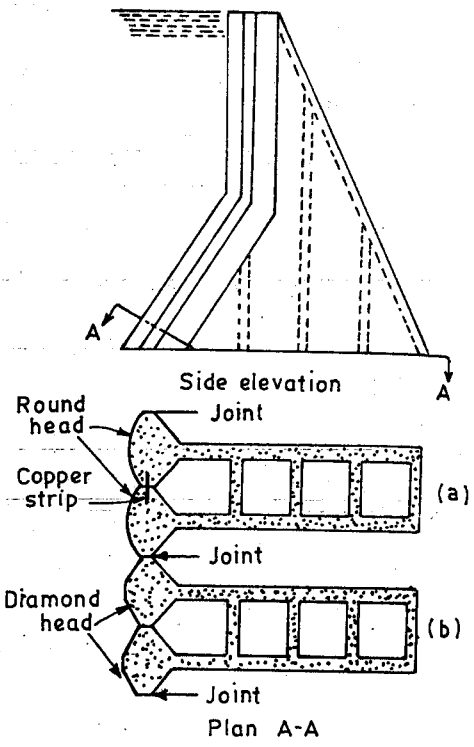


Fig. 27.15. Massive head type of buttress dam.

These *massive head buttress dams* are, infact, considered as improvement over all other types of buttress dams, as they offer, several **advantages**, such as :

(*i*) The construction work is easier, as mass concrete can be laid together in the entire dam body.

(*ii*) Since water pressure acts radially in a cylindrical type, and perpendicularly in a diamond type, all pressures are normally *compressive.* The bending as well as diagonal tension in the upstream part of the dam are, thus, absent in this type of dams.

(*iii*) Since the deck is not to be reinforced, there is no question of its failure by rusting of steel, as may happen in an ordinary slab type of buttress dam.

(*iv*) For smaller heights, they prove to give more economical buttress spacing, as compared to other two types of buttress dams.

(*v*) Such a dam body offers more resistance to sliding, because, it is considerably heavier, and has a greater sectional area along the horizontal planes.

27.4.3.3. Columnar buttress type. As shown in Fig. 27.16, this type is a modification of the ordinary slab type, as the deck here, is supported on columns. Very few dams have been built, since they offer the following *disadvantages*:

(*i*) they require very strong and stable foundations.

(*ii*) more skill is required in constructing the buttresses ; and even then, they cost practically as much as other conventional types.

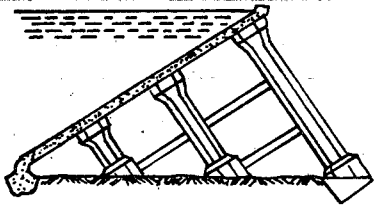


Fig. 27.16. Columnar buttress dam.

PROBLEMS

1. (*a*) Define an arch dam and a double arch dam. What are the important types of arch dams, and what are their major differences ?

(*b*) What is meant by 'the best central angle of an arch dam', and what is its value ?

2. (*a*) Contrast the design criteria for a gravity dam and that for an arch dam by the cylinder theory. What are the relative advantages of constant radius, constant angle, and variable radius dams ?

(*b*) What are the considerations for the choice of a buttress dam in preference to other types ? When do we go in for the massive head type ?

(*c*) What is meant by the following :

(*i*) Economic buttress spacing

(*ii*) Best central angle of an arch dam.

3. (*a*) Distinguish between the constant radius and constant angle layouts of an arch dam. Obtain the value of the best central angle for the latter.

(*b*) Enumerate the different types of buttress dams ; and explain as to how a slab type of buttress dam differs in its design, as compared to a concrete gravity dam.

4. (*a*) State the condition under which you would recommend the construction of an arch dam ; and explain the various methods of layout of an arch dam.

(*b*) How do you ascertain the absolute economic value of buttress spacing for an ordinary buttress dam ?

5. State the conditions under which you would recommend the construction of an 'arch dam'. Draw neat sketches of various types of arch dams and state the circumstances under which you would recommend them.

6. (*a*) What is the 'master curve' as used in connection with buttress dams ? What is its use, and how is it drawn ?

(*b*) What are multiple arch dams, and how do they differ from ordinary arch dams and double arch dams ? Draw line sketches for all these types of dams.

7. (*a*) Massive head type of buttress dams are considered to be the best of all the available types of buttress dams. Why ?

(*b*) Enumerate the design methods or theories, that are used in the design of arch dams. Which of these theories or methods, is used to compute approximate result, and which one is used for obtaining precise results. Explain any one of these in nut-shell.

8. (*a*) Differentiate between 'introdos' and 'extrodos', as are applicable in arch dams.

(*b*) What are double curvature arch dams, and how do they differ from arch dams ?

(*c*) What is the difference between a 'constant centre arch dam' and a 'constant angle arch dam' ?

(*d*) What is meant by economic spacing of buttresses ; and how is it related to dam height and upstream slope of buttress dam ?

9. Write short notes on any four of the following :

(*i*) arch dams.

(*ii*) buttress dams.

(*iii*) methods of layout of an arch dam.

(*iv*) design principles involved in arch dams.

(*v*) master curve for economic buttess spacing.

(*vi*) best central angle for an arch dam.

(*vii*) design criteria for a slab type of buttress dam, and its comparison with the design of a gravity dam.

(*viii*) massive head type of buttress dams.

(*ix*) multiple arch dams.

(*x*) double curvature arch dams.

28

Irrigation Revenue Rates

28.1. Methods of Pricing Irrigation Water

When water is supplied to the cultivators for irrigating their crops, then naturally, it has to be charged and priced at some specified rate. This rate may be based either : (*i*) on flat rate basis ; or (*ii*) on the basis of the actual quantity of water consumed by each cultivator, as discussed below :

28.1.1. Flat Rate Pricing. When pricing of irrigation water is done on flat rate or crop rate basis, then the rates are specified per hectare of each crop sown and cultivated by the cultivators. Different rates for different crops are generally specified, depending upon the water requirement of the crop.

In other words, a crop requiring the highest quantity of irrigation water, such as sugarcane or Banana, will be priced at the highest rate ; whereas the crop requiring the lowest quantity of water, such as Paddy or Groundnut, shall be rated at the lowest rate. Different crops will, thus, generally be rated at reducing rates, with their reducing irrigation requirements, such as shown in Table 28.1 (*a*).

Table 28.1. (*a*) Water Requirements, and Specified Irrigation Water Rates for Different Crops in Maharashtra State of India (1990-data)

S.No.	*Name of the crop sown*	*Water required as depth of irrigation (Δ) in cm*	*Vol. of water required per ha of crop in m^3*	*Specified Irrigation Revenue Rate in Rs/ha of each crop.*
(1)	(2)	(3)	(4)	(5)
1.	Sugar-cane (*perennial*)	270	27,000	750/–
2.	L.S. Cotton (*Kharif-Ravi*)	90	9,000	400/–
3.	Vegetables (*Ravi*)	75	7,500	—
4.	Wheat (*Ravi*)	45	4,500	75/–
5.	H.Y. Maize (*Kharif*)	45	4,500	—
6.	H.Y. Jowar (*Kharif*)	22	2,200	—
7.	Paddy (*Kharif*)	20	2,000	50/–
8.	Groundnut (*Kharif*)	15	1,500	300/–

It can be seen from Col (5) of the above table that although the specified revenue rates are generally inaccordance with the general pricing formula of specifying higher rates for higher water requirement, yet there is a deviation in the case of ground-nut crop, which is priced at six times the rate of paddy, although its water consumption is somewhat lesser than that of paddy. One of the reasons for pricing groundnut at higher rate is the *higher earning capacity* of this crop, as compared to that of paddy, as shown in Col. (7) of table 28.1 (*b*).

Table 28.1. (*b*) Earnings provided by different crops in Maharashtra State (1990-Data)

Sl. No.	*Name of crop*	*Yield per ha in Qtl (100 kg) Qtl/ha*	*Sale price in Rs./Qtl*	*Gross income per ha in Rs./ha (3)×(4)*	*Estimated cost of cultivation (without accounting for irrigation charges in) Rs/ha*	*Net income in Rs/ha (5)—(6)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
1.	Sugar-cane	865	30	25,950	12,450	13,500 (Max.)
2.	L.S. cotton	19	498	9462	5,002	4,460
3.	Vegetables	198	110	21,780	10,325	11,455
4.	Wheat	25	290	7,250	3,640	3,610
5.	H.Y. Maize	30	185	5,550	2,500	3,050
6.	H.Y. Jowar	27	200	5,400	2,530	2,870
7.	Paddy	37	215	7,955	4,644	3,311
8.	Ground nut	17	575	9,775	4,854	4921

Note. A comparison between Col. (5) of table 28.1 (*a*) and Col (7) of table 28.1 (*b*) will reflect that the specified irrigation revenue rates for different crops are varying between 2 to 9% of their earning capacities. It also reflects that the rates are highly beneficial to sugarcane, which is a highly earning crop.

It can, therefore, be concluded that the specified rates in a *crop rate system of pricing,* mainly depends upon two factors ; viz.

(*i*) *water requirement of each crop* ; and

(*ii*) *earning capacity of that crop.*

28.1.1.1. Demerits of the crop rate system. This method does not offer any incentive to the cultivators for economising in the use of water, since they will be charged at flat rate for the area of the crops grown. The cultivators will, therefore, never bother to avoid wastage of water, and will also try to use more and more quantity of water, under the wrong notion that more water will flourish their crops more.

This method, therefore, may lead to over-utilisation of water, particularly in command areas falling in the head-reaches of canals. *This may ultimately lead to water-logging and salinity of the irrigated lands, besides leading to the wastage of the precious and scarce irrigation water, which will ultimately cause scarcity of water at the tail reaches of the canal command, thus leading to unequitable distribution of water in the command area of the irrigation system.*

28.1.1.2. Fixed Assessment of grown crops in crop rate revenue system. The field assessment of the area of each crop grown by each cultivator is generally carried out and recorded by the area *Patwari* at the time of sowing, and also at the time of maturity and harvesting. At the end of the crop period, a *demand statement,* called **Khatauni,** for each cultivator is prepared.

The recorded measurements of crops carried out by Patwaris, are generally test checked by officers above Patwaris, such as *Tehsildars (Ziledars)*, SDM, *ADM (Revenue)*, etc, working under the Revenue Department of the Government. In States, where the engineers are also designated as canal officers, such powers are also vested in the hands of sub-divisional and divisional engineers. Checking of irrigated areas, thus, constitute an important part of the duties of engineers. In such cases, divisional engineers (*i.e.* executive engineers) will also have to raise the revenue bills against the cultivators by signing the Khatuni's on 31st June and 31st Dec. every year, for Ravi and Kharif crops, respectively.

In case of failure of crops or reduced production, due to reasons beyond the control of cultivators, full or part remissions can be granted by canal officers. Additional fines and levies may also be imposed by canal officers on cultivators, in case of unauthorised water use, or for damaging the outlets, or cutting the canal banks, or even for wasting water. The decision of the divisional canal officer is generally final and binding in all such cases, because his decisions can be appealed against only at a very high level, *i.e.* to the commissioner (Revenue division), who is normally a senior IAS officer above the Deputy Commissioner of the District.

28.1.2. Volumetric Method of Pricing. In this method, pricing of irrigation water is done on the basis of actual consumption of water by each cultivator, or by a chunk of cultivators, such as by a cooperative society of the cultivators.

This can be done only when measuring devices are installed at the outlets of irrigation minors or water-courses. The method, therefore, requires costly installations, like those of *water-meters* or *venturi-meters,* or *modular outlets.*

Due to the high installation and maintenance cost of such measuring devices as required in this system, this method of revenue assessment has generally not been adopted in our country. This method, therefore, although has its inherent advantages leading to economic use of water, does not find much practical use in India.

The method can be made use in assessing canal irrigation revenues, only if, Irrigation Departments supply water to cooperatives or Panchayats rather than dealing with the individual cultivators. If this can be achieved, then naturally, very less number of meters would be required and this system of charging can be implemented.

This method of pricing irrigation water, can of course, be used much easily in *tubewell irrigation,* where the billing can be done on the basis of number of units of electrical energy consumed in running a particular tubewell-supplying water to a particular chunk of cultivators. This method of charging by the number of electrical energy units consumed or by the number of running hours of the tubewell, is largely used where private tubewells supply water to the cultivators, although its use on Govt. irrigation supplies is still not being adopted on a large scale. The States have simply specified slightly different rates for lift irrigation supplies, and tubewell supplies are covered under those rates.

28.1.2.1. Merits and demerits of volumetric method. This pricing method offers its best merit of economical use of water, thus, avoiding the wastage of this preciously scarce commodity, and also avoiding water-logging and salinity of irrigated lands.

The *demerits* of this method are that :

(*i*) it requires costly measuring devices, atleast at each outlet head, and round-the-year-staff to maintain and look after these devices ;

(*ii*) it promotes unauthorised drawals of irrigation supplies, and also manipulation and mishandling of the measuring devices at the outlet heads ;

(*iii*) it forces the chunk of cultivators to pay equally in accordance with the water supplied at the outlet head, although the cultivators at the tail end of the water-course gets lesser supplies due to wastage of water enroute the length of the water course ; and

(*iv*) silt debris may retard the flow through the metered outlets.

28.2. Economic Water Rates Vs Prevailing Revenue Rates in India

We have stated earlier that in our country, the Govt revenue rates for irrigation waters are generally fixed by considering ; (*i*) *The water requirement of the crop* ; and (*ii*) *the earning capacity of the crop.* However, the most important factor, *i.e. the real cost of the irrigation water,* is generally being ignored.

This real cost of the water can be worked out on *Economic considerations,* by computing the cost of the irrigation project, and recovering the same during its effective life span, along with costs of interest and annual operating and maintenance charges of the irrigation system. Such a rate, when worked out in terms of Rs. per unit quantity of water obtained, is known as the *economic water rate.*

Such a rate will evidently depend upon *the cost of the irrigation project, its effective life span, the rate of interest considered on the investment, and actual O and M charges on the system.* The *annual recovery factor,* computed on Economic considerations as explained in chapter 12 of "Hydrology and Water Resources Engg"., can, thus, give us the cost of irrigation water per year from a particular project, and it can finally be used to work out the unit cost of such water.

Such economic water rates have been computed to be generally much higher than the rates being changed by the government bodies in the country.

For example, economic water rate on a typical tubewell minor irrigation project, in Gujarat State has been computed to be Rs. 0.30P per kilolitre (on 1990 rates) as against the rate being charged by the State Tubewell Corporation @Rs. 0.18P per kilo litre. Similarly, for a typical interstate minor lift irrigation scheme, executed in Gujarat and Orissa States, the economic rate has been worked out to be Rs. 1.53P per kilolitre (on 1990 rates), as against the rate of Rs. 0.18P per kilolitre being charged by the Gujarat State Corporation, and Rs. 0.02P per kilolitre being charged by the Orissa State.

It has thus, generally been seen that the rates, being charged by the Govt. Corporations or State Irrigation Departments, are *highly subsidised* and *too low* incomparison to the actual cost of water. *So much so, that the rates being charged are not even capable to repay the O* and *M charges alone, what to talk of recovering the capital investment on the project.*

Considering the urgent necessity of improving the rates, so as to reduce the heavy losses and subsidies, being incurred by the Government in the operations of irrigation systems, the newly framed *National Water Policy (1987),* has specified that :

"The irrigation water rates should be such as to convey the scarcity value of resources to the users, and to fasten the motivation for economy in water use. *The revenue rates should, therefore, be adequate to recover the annual operation and maintenance charges and a part of the fixed capital cost.* Efforts should be made to reach this ideal, over a period of time, while ensuring the assured and timely supplies

of irrigation water. The water rates for surface water and ground water should be rationalised with due regard to the interests of small and marginal farmers".

From the above, it can be seen that the built in requirement of the National policy is that the water rates should recover annual R and M charges and part of the capital cost, if not the full cost. Even if for the time being, we neglect recovering of the capital cost, we must rationalise our irrigation revenue rates in such a fashion as to fully recover our annual running and maintenance expenditure on the irrigation systems.

It has, however, been computed that the average annual irrigation revenue realisation in the country is only about 12% of the total O and M expenditure alone, with Statewise variation ranging between 32% in Delhi to 0.11% in Nagaland with Punjab not collecting any revenue at all, since it has totally abolished water charges for irrigation water supplies wef Feb. 1997.

There is, therefore, an urgent need to reconsider and revise the water rates, even at the cost of inflation and possible rise in the rates of crops.

At present, different revenue rates are prevailing in different States of the country even for the same crop ; and even in the same State, different rates are prevailing for different irrigation systems. There is no uniform method of computing the revenue rates ; and the rates are generally being decided and guided by the whims and fencies of the political powers. A few examples of the non-uniformity and irrationality prevailing in our present revenue rates, are highlighted below :

In Gujarat State, the revenue rates for lift irrigation on canals are $\frac{1}{2}$ times, and on rivers and reservoirs $\frac{1}{3}$rd times the flow irrigation rates, provided the water is lifted by the cultivators at their own cost. In Maharashtra, the water rates for all types of lift irrigations to $\frac{1}{10}$th times (when cost of lifting is met by cultivators) are hardly $\frac{1}{5}$th to $\frac{1}{10}$ times the rates prescribed for flow irrigation. In Haryana State, the rates are reduced only to $\frac{1}{2}$ times of the flow irrigation rates, when water is privately lifted from any State irrigation work. In Jammu and Kashmir State, the rates prescribed for lift irrigation (cost of lifting is met by Govt.) are about 15 times the rates prescribed for flow irrigation ; whereas, in Kerala, the prescribed rates for such Govt. lift irrigation schemes are only about $1\frac{1}{2}$ times the rates for flow irrigation. States of Andhra Pradesh, Assam, Bihar, Chhatisgarh, Jharkhand, Karnataka, Madhy Pradesh, Tamil Nadu & West Bengal have not prescribed any separate rates for lift irrigation, and flow irrigation rates remain applicable to lift irrigation rates also. The State of Punjab has completely abolished the water rates wef 14.2.1997. While Himachal Pradesh has levied extremely low rates — lowest amongst all the States. Besides Punjab, some other States like Arunachal Pradesh, Sikkim, Meghalya, Mizoram, and UT's like Andaman and Nicobar Islands, Chandigarh and Lakshadweep have also not levied any irrigation revenue rates, so far, although the States of Sikkim and of Arunachal Pradesh are in the process of introducing such rates.

While the rich-farmers State like Punjab has completely abolished the irrigation revenue rates, the progressive States of Maharashtra and Gujarat have not only revised their water rates in the year 2001, but have also made an inbuilt provision of automatic

annual increase in their notified rates. Say for example, the Maharashtra State has specified a uniform 15% annual increase over the rates notified on 1.9.2001. Similarly, the Gujarat State has provided for 25% annual increase in notified water rates of Paddy, and 15% annual increase in rates notified for all other crops wef 16.2.2001.

It can also be seen from Table 28.2 that certain States and UT's like Andhra Pradesh, Delhi, Goa, Himachal Pradesh, J & K, Karnataka, Kerala, Manipur, West Bengal, etc. have not revised their irrigation revenue rates for the last more than 10 to 50 years. NCT of Delhi is in worst condition in this respect, as it has not revised its rates since the year 1951. There is an urgent need for revision of the water rates in all such States.

Table 28.2. Table Showing the Dates on Which Irrigation Revenue Rates were Last Notified in Various Indian States

State	Date on which irrigation revenue rates were last notified	State	Last date on which irrigation revenue rates were notified
Andhra Pradesh	7.1.1196	Maharashtra	1.9.2001
Assam	30.3.2000	Manipur	Year 1977-78
Bihar	Nov. 2001	Mizoram	Not specified
Chhattisgarh	15.6.1999	Orissa	5.4.2002 for flow irrigation July 1997 for lift irrigation
Delhi	Year 1951	Punjab	Abolished wef 14.2.1997
Goa	11.2.1988	Rajasthan	24.5.1999
Gujarat	16.2.2001	Tamil Nadu	6.11.1987
Haryana	27.7.2000	Tripura	Yet to start
Himachal Pradesh	Year 1977	U.P.	18.9.1995
J & K	1.4.1976	Uttaranchal	18.9.1995
Jharkhand	14.11.1995	West Bengal	6.4.1997
Karnataka	1.7.1985	Andaman and Nicobar Island	1.11.1992
Kerala	18.9.1974	Dadar and Nagar Haveli	29.1.1996
Madhya Pradesh	15.6.1999	Daman and diu	Year 1980

The prevailing irrigation water rates in different States of the country, for flow irrigation are compared in Table 28.3; while those for lift irrigation are compored in table 28.4.

With a view to rationalise the irrigation water rates in different states, the planning commission, Govt. of India, appointed a "*Committee on Pricing of Irrigation Water*", vide notification dated 23-10-1991, under the chairmanship of Dr. A. Vaidyanathan, Professor of Madras Institute of Development studies and former Member of Planning Commission, with 16 members from Central Water Commission (CWC) and Other State Governments. This committee has submitted its report in September, 1992. But the same appears to have not yet been implemented. Some of the important recommendations of this Committee are briefly pointed out in article 28.3.

Table 28.3. Comparison of Prevailing Irrigation Revenue Rates (Maximum) in Various States of India for Flow Irrigation

Water rates for flow irrigation in Rs. per hectare of each crop grown

S.No.	Name of State	*Rice or paddy (Kharif)*	*Sugarcane*	*Wheat*	*Cotton*	*Oil seeds*							*Barley*	*Maize*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1.	Andhra Pradesh	494	494	—	494	494	494	—	—	—	—	—	—	—
2.	Assam	281	222	563	—	99	—	—	—	150	—	—	—	—
3.	Bihar	217	371	185	—	247	99	148	—	99	—	99	148	—
4.	Chhatisgarh	494	741	200	371	247	247	494	247	247	494	247	494	—
5.	Delhi (NCT)	99	—	45	—	30	—	89	—	—	—	—	44	30
6.	Goa	150	300	—	—	100	—	100	100	—	—	—	—	60
7.	Gujarat*	300	2750	240	275	440	130	220	440	—	330	90	—	90
8.	Haryana	148	197	124	124	124	99	148	86	—	148	99	124	86
9.	Himachal Pradesh	24	41	7	17	16	12	8	—	—	21	11	16	14
10.	Jammu and Kashmir	49	49	25	—	25	20	37	—	—	—	—	—	25
11.	Jharkhand	109	371	185	—	99	99	185	—	99	—	99	185	—
12.	Karnataka	247	988	148	148	—	—	—	148	—	87	—	—	87
13.	Kerala	99	—	—	—	—	—	—	—	—	—	—	—	—
14.	Madhya Pradesh	200	741	200	371	—	247	494	247	—	494	247	494	—

* With annual increase of 25% for Paddy and 15% for other crops since 16.2.01.

Contd....

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
15.	Maharashtra*	180	4763	360	280	548	—	270	548	—	—	—	—	—
16.	Manipur	38	—	38	—	23	23	23	—	—	—	—	—	—
17.	Orissa	—	500	170	—	170	170	280	170	84	420	—	—	140
18.	Punjab	Charges abolished wef 14.2.1997												
19.	Rajasthan	114	247	148	178	114	79	109	—	—	178	182	79	67
20.	Tamil Nadu	37	49	—	62	—	—	50	—	—	49	—	—	—
21.	Tripura (Proposed rates)	—	—	313	—	—	313	313	313	—	—	—	—	—
22.	Uttar Pradesh	287	474	287	114	—	—	287	—	—	306	—	287	—
23.	Uttaranchal	143	—	143	—	—	—	—	—	—	153	—	—	—
24.	West Bengal	37	—	49	—	—	—	—	—	—	—	—	—	—
25.	U. Ts													
	(*a*) Dadra and Nagar Haveli	140	830	—	—	—	—	110	—	—	—	—	—	—
	(*b*) Daman and Diu	200	—	—	—	—	—	—	—	—	—	—	—	—

* With annual increase of 15% wef 1.9.01

Note. States which have not levied any irrigation revenue charges so far, have not been included in this table.

Table 28.4. Comparison of Prevailing Irrigation Water Rates (Maximum) in Various States of India for Lift Irrigati

Water rates for Lift irrigation in Rs per hectare of each cnp grown

S.No.	Name of State	*Rice or paddy (Kharif)*	*Sugarcane*	*Wheat*	*Cotton*	*Oil seeds*	*Pulses*	*Vegetables*	*Groudnut*	*Jute*	*Tobacco*	*Gram*	*Barley*	*Maize*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1.	Andhra Pradesh*	494	494	—	494	494	494	—	—	—	—	—	—	—
2.	Assam*	281	222	563	—	99	—	—	—	150	—	—	—	—
3.	Bihar*	217	371	185	—	247	99	148	—	99	—	99	148	—
4.	Chhatisgarh*	494	741	200	371	247	247	494	247	247	494	247	494	—
5.	Delhi (NCT)	148	—	67	—	45	—	133	—	—	—	—	67	44
6.	Goa	300	600	—	—	—	120	200	200	—	—	—	—	120
7.	Gujarat**	170	420	100	90	180	55	90	90	—	—	45	113	35
8.	Haryana	74	99	62	62	—	49	74	43	—	74	49	62	43
9.	Himachal Pradesh	49	82	29	32	32	24	32	—	—	41	29	32	28
10.	Jammu & Kashmir	247	716	124	—	—	—	—	—	—	—	—	—	—
11.	Jharkhand*	109	371	185	—	99	99	185	—	99	—	99	185	—
12.	Karnataka*	247	988	148	148	—	—	—	148	—	87	—	—	87
13.	Kerala	149	—	—	—	—	—	—	—	—	—	—	—	—
14.	Madhya Pradesh*	200	741	200	371	—	247	494	247	—	494	247	494	—

Contd....

* Separate rates for Lift irrigation are not given, and hence rates shown are the same as for flow irrigation.

** Specified rates here are for lift irrigation done by cultivators at their own cost from canals & irrigation projects.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
15.	Maharashtra	—	495	—	100	—	—	—	100	—	—	—	—	—
16.	Manipur	38	—	38	—	—	23	23	—	—	—	—	—	—
18.	Mizoram	No water rates are in Vogue												
17.	Orissa	516	1398	646	861	345	215	1292	684	430	1034	—	—	—
18.	Punjab	Charges abolished wef 14.2.1997												
19.	Rajasthan	395	573	296	356	227	158	217	—	—	356	207	158	133
20.	Tamil Nadu*	37	49	—	62	—	—	50	—	—	49	—	—	—
21.	Tripura (Proposed rates)	—	—	—	—	—	—	—	313	313	—	—	—	—
22.	Uttar Pradesh	143	237	143	57	—	—	143	—	—	153	—	143	—
23.	Uttaranchal	287	—	287	—	—	—	—	—	—	306	—	—	—
24.	West Bengal*	37	—	49	—	—	—	—	—	—	—	—	—	—
26.	U. Ts													
	(*a*) Dadra and Nagar Haveli	75	100	100	—	—	100	275	—	—	—	100	—	—
	(*b*) Daman and Diu*	200	—	—	—	—	—	—	—	—	—	—	—	—

* Separate rates for Lift irrigation are not given, and hence rates shown are the same as for flow irrigation.

nt Recommendations of the Committee on Pricing of Irrigation (1992)

s 3.11 and 3.12. A revision in the level and structure of water rates is, thus, necessary in the interest of both efficiency and equity. The revision should be such as to achieve full cost of recovery in due course and in the process promote saving, create disincentives for waste and thereby enable the service area to be expanded and more reliable service assured.

(2) **Paras 3.13 and 3.16.** Revision of water rates should go hand in hand with measures to improve the quality of service and to keep a check on costs.

(3) **Paras 3.54 and 3.56.** The objective of the *first phase* should be to rationalise and simplify the existing system of assessment (based on cropwise irrigated area on an individual basis) to a system of season-specific area rates. It is possible to estimate the relative water consumption per hectare irrigated in different seasons. The variable part of the tarrif in the case of major and medium projects and such of these minor works as are still under State management should be fixed on this basis. *We should urge that all minor systems be turned over to users immediately after completion.* Both categories of projects will pay a flat basic rate per ha. *The level of cost-recovery to be aimed at the first phase should at least cover the O and M costs and 1% interest on capital employed.*

(4) **Para 3.57.** Irrigated area under a crop which spreads over two seasons will be charged at the rates applicable to both seasons, and perennials for all three seasons ; but crops like paddy which take a lot of water for non-consumptive uses need specific treatment. Where paddy is a significant but not a dominant crop, some differentiation may have to be made. In each season, therefore, we need to distinguish at best three categories, viz. paddy, sugarcane, and, perennials and other crops.

(5) **Para 3.58.** In the *second phase* to be implemented in the course of the next decade, the aim would be to shift to a fully volumetric system. Additional investments to modify the distribution system for effective regulation of volume delivered at outlets (estimated at approximately Rs. 5,000 crores) will be needed. As system efficiency and productivity improve, the targets of cost recovery can be progressively increased.

(6) **Paras 3.59 and 3.61.** There are many important matters of detail to be decided in shifting to the volumetric system of charging. These are best decided in consultation with user's representatives. *The most crucial and also the most difficult task in this phase will be to promote the creation of sufficiently large farmer's groups.*

(7) **Para 3.62. Phase III,** which will spread over a much longer period should seek to extend and consolidate the system of farmer group management, and implement, with the involvement and participation of such groups, a programme for upgrading the system to a higher level of efficiency in water use and therefore of productivity. Besides substantial investments in conjunctive use and distribution networks, the techniques of water management will have to become tighter and more sophisticated.

(8) **Para 4.5.** There is a case for earmarking the whole or a substantial part of the receipts from each irrigation system towards the operation and maintenance of that system. *In the long run, there is a case for moving towards the conversion of each irrigation system into an independent self-financing system, whether through the formation of corporations or otherwise.*

(9) **Paras 4.9 and 4.10.** We recommend that States set up special expert groups to work out appropriate norms and a procedure for periodic monitoring and updating for different agro-climatic regions and broad categories of projects.

(10) **Para 4.26.** The States should form a high-powered autonomous Board which may be called "Irrigation and Water-Pricing Board" to review the policy, establish the norms regarding maintenance costs for various components and staff costs, assess the actual expenditure in relation to these norms, and determine the parameters and criteria for revising water rates. There should be a mandatory review of all these matters every five years with an opportunity for users to present their views.

(11) **Paras 5.1 and 5.2.** Among the various problems faced in the matter of assessment, unauthorised irrigation and the incorrect reporting of crops & irrigated areas are the major ones. *There are also delays in raising demands. In spite of low and subsidised water rates, actual revenue recoveries are substantially below the demands. Large arrears have been allowed to accumulate and these tend to be eventually written off.* The existing mechanisms for preventing unauthorised, excessive and wasteful use of water as well as for the recovery of outstanding dues have not proved very effective. Lack of coordination among different agencies involved in assessment and collection also aggravates the problem.

(12) **Paras 5.9. and 5.15.** There is considerable diversity in the mechanism for the assessment and collection of irrigation revenues. The limited data that we have seen suggest that the ratio of accumulated arrears to annual demand is generally much higher in States where the Irrigation Department is responsible for both the assessment and collection than in States where both functions are vested in the Revenue Department or where they are divided between the Irrigation and Revenue Departments. *Having considered the matter, the committee is of the view that the assessment function is best entrusted to the Irrigation Department. As for collection, States may choose one of two options — (1) entrusting both assessment and collection to the Irrigation Department, and (2) making the Irrigation Department responsible for assessment and the Revenue Department for collections — in the light of their specific circumstances and experience. Where alternative (1) is preferred, it would be necessary to empower the Irrigation Department officials to recover arrears of irrigation dues under the Revenue Recovery Laws.*

(13) **Paras 5.16 and 5.18.** The Committee would like to emphasise the need for purposive and strong measures to ensure the accurate assessment of irrigation charges and their prompt and full collection. We suggest that a regular system of independent verification of actual irrigation on a sample basis be introduced on all major and medium project commands. At the same time, we strongly recommend that a serious effort be made by the Irrigation Departments to use remote sensing as an independent source of information on irrigated area, which can be used along with sample verification to test the veracity of records maintained by the field staff. Such independent checks linked to a system of penalties for inaccurate (and rewards for accurate) recording would minimise the loss from under-assessment.

(14) **Para 5.19.** We are of the view that the practice of waiving or suspending collections of irrigation charges on account of drought is not justified in respect of areas actually irrigated.

(15) **Paras 5.20 and 5.21.** The reluctance of the governments to support the agencies concerned in enforcing the regulations has led to a situation in which these agencies have practically given up even raising demands for betterment levies. Very little is done to take cognizance of the widespread violations of rules and even less to enforce what

little penalties are levied. We need hardly emphasise that such laxity has serious consequences not just in terms of revenue but for the efficient management of the system.

(16) **Para 5.22.** With a view to improving collections, the States should consider switching from the existing system of supplying water on credit to one of supply against advance payment. The collection performance relative to demand should be an important consideration for deciding the allocation of *O* and *M* funds to individual systems. We also recommend that proceeds from the collection of accumulated arrears from a system be used for making up the cumulative effects of past neglect in the maintenance of that system.

(17) **Paras 5.23 to 5.25.** Until a system of group assessment on a volumetric basis is introduced, the State agencies will need to verify and record the area irrigated by plots in order to determine the dues from individual farmers. The proposed system of season-hectare assessment (*i.e.*, assessment on the basis of area irrigated in each season) will substantially simplify the task. In the case of minor surface works, since assessment will be at a flat rate per hectare of command, there is no need for recording cropwise area irrigated for the assessment of water rates.

(18) **Para 5.26.** The aim should be to increase user participation in management initially at the level of distributaries and minors, and in due course at the level of the system as a whole. Each system should become an autonomous entity which manages its own finances both for operation and eventually for the expansion/improvement of facilities.

(19) **Para 7.1 to 7.6.** For the purposes of illustrating the application of the suggested approach to the revision of water rates, if we use the norm suggested by the Jakhade Committee, namely, Rs. 180 per ha. gross irrigated area, with adjustments for inflation since 1987 and for departmental overheads at 25 per cent of the norm so adjusted, and add interest @ 1% on capital, *the total recovery in Phase I should average around Rs. 340 per ha.* As against this, the estimated gross receipts from major and medium projects in 1989-90 was Rs. 68 ; the actual irrigation revenue works out in 1989-90 to Rs. 50 per ha. Assuming conservatively the additional revenue on account of an increase in the rates for industrial use at Rs. 10 per ha, **the recovery from irrigation charges in Phase-I has to be Rs. 310 per ha compared to the present realisation of Rs. 50 per ha.**

(20) **Para 7.9.** A basic levy at the rate of Rs. 50 per ha is recommended for all lands in the cultivable commands of major and medium as well as minor works. This is intended as a fee for the right to get water from the system (a sort of "demand charge").

(21) **Para 7.10 to 7.12.** The rate per unit of water needs to be equalised across crops. The additional revenue through such rationalisation will be sizeable, the increase ranging from 18 per cent to 140 per cent of the revenues at current rates if the per ha. cm. rates for all crops are made equal to the irrigation rate now charged for ID crops, and from 50 to 325 per cent if they are made equal to the highest irrigation rate per ha.cm.

(22) **Para 7.13.** The level of rates will also have to be raised. The extent of increase required, depending as it does on the potential for rationalisation, cannot be quantified. It is also likely to vary from States to States. Nevertheless, on the average, the required revenue by way of irrigation charges (Rs. 310 per ha) will still be barely 6% of the gross produce per hectare of the irrigated area, and that without taking any account of likely improvements in productivity.

(23) **Para 7.15.** As a measure of inducement for farmer's groups to take over greater responsibility, we suggest that when the proposed revisions are implemented, the rates

for group delivery be fixed at substantially lower levels than for individual delivery, while keeping the basic fee of Rs. 50 per ha. common.

(24) **Para 7.22.** The implementation of the approach suggested here will require expeditious action on the part of each State to set up task forces, with adequate expert staff and authority for collecting the necessary data, to determine *O* and *M* norms by region and category of projects ; undertake sample studies in the field to determine the extent of under-assessment and under-collection at existing rates ; determine the per hectare rates applicable to paddy and other seasonal crops by season and for perennials in terms of volume of irrigation required and costs connected with carry-over between seasons ; and work out the existing and projected use by non-agricultural users and determine the rates to be charged to such users, the appropriate contractual arrangements, and other relevant details.

The committee report, however, appears to be lying in the waste paper baskets, of the States, and the States are simply not inclined to take politically inconvenient decisions of increasing the water rates. Let us appeal to the politicians of this country to forget their political benefits infront of the overall benefits of the country.

PROBLEMS

1. Discuss the two important methods, which are used to price the irrigation water supplies for collecting irrigation revenues. Compare the merits and demerits of both these methods. Also discuss as to which of these two methods is generally being used in our country and why.

2. (*a*) Explain the various points which need to be considered while fixing irrigation water rates. *(Pune Univ. May 1995)*

(*b*) Explain the crop rate basis of assessment of water charges. State the other method. *(Pune Univ. Nov. '95)*

3. (*a*) Enumerate the merits and demerits of volumetric assessment of water charges. *(Pune Univ. Nov. '96)*

(*b*) How have the irrigation revenue rates been fixed in India ?

Compare the irrigation revenue rates prevailing in different States of India, explaining the urgent necessity of revision of these rates. What has the GOI done to effect this revision and/or to switch to volumetric assessment system.

4. (*a*) "The revenue rates for canal irrigation water should differ from the revenue rates for lift irrigation supplies". Critically discuss this statement in special reference to Indian States.

(*b*) Point out the major recommendations of the Expert Committee, constituted by GOI in 1992, for revision of irrigation revenue rates.

—29

Chapterwise Multichoice Objective Questions

CHAPTER -1

Q. 1. Irrigation is basically required in :

(*a*) humid regions (*b*) arid regions
(*c*) semi arid regions (*d*) all of the above.

Q. 2. Irrigation may not be needed in :

(*a*) England (*b*) Israel
(*c*) India (*d*) none of the above.

Q. 3. Pinpoint the correct statement :

(*a*) irrigation helps in adopting mixed cropping
(*b*) 'mixed cropping' means sowing of a different crop after a particular crop has been grown
(*c*) over-irrigation may lead to saving in fertilizers
(*d*) irrigation helps in avoiding mixed cropping.

Q. 4. 'Flood irrigation' method of irrigating fields, works best on :

(*a*) level or gently rolling terrain
(*b*) steeply rolling terrain
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 5. In flood irrigation, the preferred method of applying irrigation water to the comparatively steeper rolling land, is :

(*a*) check flooding (*b*) boarder flooding
(*c*) wild flooding (*d*) basin flooding.

Q. 6. Fields enclosed by dikes, where rice is grown, are called :

(*a*) paddies (*b*) furrows
(*c*) basins (*d*) none of the above.

Q. 7. The method of growing crops on ridges, running on the sides of water ditches, is known as :

(*a*) flood irrigation (*b*) furrow irrigation
(*c*) check irrigation (*d*) none of them.

Q. 8. In a field under furrow irrigation, 'furrows' are referred to represent:

(*a*) ridges on which crops are grown
(*b*) narrow ditches which carry irrigation water
(*c*) both (*a*) and (*b*)
(*d*) neither (*a*) nor (*b*).

Q. 9. In a mildly water scarce area, the drip irrigation could be preferred for growing :
(*a*) wheat (*b*) fodder
(*c*) rice (*d*) fruits and vegetables.

Q. 10. The most suitable water for irrigation is :
(*a*) C 1—S 1 (*b*) C 2—S 2
(*c*) C 4—S 1 (*d*) C 1—S 4.

Q. 11. Addition of gypsum to the irrigation water is recommended to overcome difficulties posed by :
(*a*) highly saline irrigation supplies
(*b*) irrigation supplies containing high quantities of sodium
(*c*) irrigation supplies containing heavy sediment
(*d*) all of the above.

Q. 12. The electrical conductivity of medium saline water (C 2) at 25°C is of the order of :
(*a*) 50 to 100 μΩ/cm (*b*) 100—250 μΩ/cm
(*c*) 250 to 750 μΩ/cm (*d*) 750—1500 μΩ/cm.

Q. 13. The Sodium Absorption Ratio of an irrigation water is 8. This water will be called :
(*a*) low sodium water (*b*) medium sodium water
(*c*) high sodium water (*d*) none of the above.

Q. 14. High sodium content in irrigation water is :
(*a*) generally good
(*b*) generally bad
(*c*) generally good, but bad for a few crops
(*d*) generally bad, but good for a few crops.

Q. 15. Salinity in irrigation water is measured by :
(*a*) SAR value (*b*) Electrical-conductivity value
(*c*) pH-value (*d*) none of the above.

Q. 16. The time required to irrigate a strip of area 0.203 hectare by a stream discharge of 0.043 cumec, to provide an average depth of 6.35 cm to the field, is : (assume average rate of infiltration to be 5 cm/h)
(*a*) 2.75 hour (*b*) 1.35 hour
(*c*) 1.5 hour (*d*) 1.90 hour. *(Calcutta University)*

Q. 17. If the concentration of Na, Ca, and Mg in a water sample are 345, 60 and 18 mg/1, respectively, then the Sodium Absorption Ratio (SAR) of this water will be :
(*a*) 5 (*b*) 10.
(*c*) 39 (*d*) 55.

CHAPTER 2

Q. 18. With the increase in supplied irrigation water, the yield of crops:
(*a*) increases continuously
(*b*) decreases continuously
(*c*) increases up to a certain limit, and then becomes constant
(*d*) increases up to a certain limit, and then decreases.

Q. 19. The most expected crops in a hot arid district of Rajasthan State in India, in the month of September, are :
(*a*) Rice and Sugarcane (*b*) Bazra and Maize
(*c*) Wheat and Maize (*d*) Tobacco and Cotton.

Q. 20. The maximum irrigation requirement of Rice crop is exhibited by its :
(*a*) maximum delta value (*b*) maximum duty value
(*c*) minimum duty value (*d*) none of the above.

Q. 21. The crop among the following, which is expected to have the maximum duty, is
(*a*) Wheat (*b*) Rice
(*c*) Sugarcane (*d*) Cotton.

Q. 22. Kor-Watering is the irrigation water supplied to a crop :
(*a*) at the time of its sowing
(*b*) just before harvesting
(*c*) about three weeks after sowing
(*d*) about three weeks before harvesting.

Q. 23. The duty of irrigation water for a given crop is maximum :
(*a*) on the field
(*b*) at the head of the main canal
(*c*) at the head of the water-course
(*d*) none of them.

Q. 24. The important Govt. reference made to the 'duty for a crop,' is usually related to its duty :
(*a*) on the field
(*b*) at the head of the main canal
(*c*) at the head of the water course
(*d*) none of them.

Q. 25. The duty at the end point of a canal minor, where the Govt. control usually ceases, is called :
(*a*) duty on field (*b*) outlet duty
(*c*) flow duty (*d*) storage duty.

Q. 26. The first important watering of crops is usually called :
(*a*) paleo watering (*b*) kor-watering
(*c*) crop-watering (*d*) all of the above.

Q. 27. The crop sequence, which cannot serve any useful purpose in 'Crop rotation', is :
(*a*) Wheat—Juar—Gram (*b*) Rice—Gram—Rice
(*c*) Cotton—wheat—Gram (*d*) Rice—Wheat—Cotton.

Q. 28. The optimum depth of kor watering for rice is about :
(*a*) 25 cm (*b*) 19 cm
(*c*) 13.5 cm (*d*) 9 cm.

Q. 29. The kor period, within which a crop must receive its first major watering, will be :
(*a*) less for humid climates (*b*) less for dry climates
(*c*) equal for all climates (*d*) none of the above.

Q. 30. The crop, out of the following, which is not a cash crop, is :
(*a*) Jute (*b*) Tea
(*c*) Rice (*d*) Sugarcane.

Q. 31. In India, the cultivated area under Rabi season is generally x times the area under Kharif season; where x is :

(*a*) 1 (*b*) $\frac{1}{2}$

(*c*) 2 (*d*) none of them.

Q. 32. The relationship between the duty D in ha/cumecs, the water depth Δ in cm, and base period B in days, is given by :

(*a*) $D = \frac{864\,B}{\Delta}$ (*b*) $D = \frac{8.64\,B}{\Delta}$

(*c*) $D = \frac{864\,\Delta}{B}$ (*d*) $D = \frac{8.64\,\Delta}{B}$

Q. 33. Consumptive use of water for a crop represents :

(*a*) The transpiration needs of the crop

(*b*) evaporation needs of the cropped area.

(*c*) evapotranspiration needs of the cropped area plus the minor quantity required in plant metabolism

(*d*) None of the above.

Q. 34. The amount of irrigation water required to fulfil the evapo-transpiration and minor metabolic plant needs per unit of cropped area is called :

(*a*) consumptive use

(*b*) consumptive irrigation requirement

(*c*) net irrigation requirement

(*d*) none of the above.

Q. 35. When there is no percolation loss for leaching and other such soil needs, then the Net irrigation requirement (NIR) of a cropped area, will be equal to :

(*a*) the consumptive use (C_u)

(*b*) $C_u - R_e$: where R_e is effective rainfall during cropping

(*c*) $\frac{C_u - R_e}{\eta_a}$: where η_a is water application efficiency

(*d*) none of the above.

Q. 36. The Gross Irrigation Requirement (GIR) of water is equal to :

(*a*) NIR + η_a (*b*) NIR + $\eta_a \cdot \eta_c$

(*c*) NIR + $\frac{\eta_a}{\eta_c}$ (*d*) none of above.

where η_a = water application efficiency

η_c = water conveyance efficiency.

Q. 37. The ratio of the water stored in the root zone of a crop, to the water actually delivered to the crop in the field, is known as;

(*a*) water conveyance efficiency

(*b*) water application efficiency

(*c*) water use efficiency

(*d*) none of the above.

Q. 38. The water which can be utilised by the crops from the soil is called :

(*a*) field capacity water (*b*) capillary water

(*c*) hygroscopic water (*d*) none of the above.

Q. 39. The moisture held by a well drained soil against gravity drainage, by the force of surface tension between the soil grains and water drops, is called :
(*a*) field capacity water (*b*) hygroscopic water
(*c*) capillary water (*d*) water of adhesion.

Q. 40. Permanent wilting point moisture content for a crop represents the :
(*a*) hygroscopic water (*b*) capillary water
(*c*) field capacity water (*d*) none of the above.

Q. 41. Available moisture for a crop is equal to :
(*a*) field capacity moisture content—Wilting point moisture content
(*b*) field capacity moisture content—hygroscopic moisture content.
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 42. The optimum moisture content (m.c.) which is retained in the root zone of a soil, before applying irrigation water, is :
(*a*) equal to : (the field capacity m.c.—wilting point m.c.)
(*b*) less than : (the field capacity m.c.—wilting point m.c.)
(*c*) more than : (the field capacity m.c.—wilting point m.c.)
(*d*) may be more or less than : (the field capacity m.c.—wilting point m.c.) depending upon the crop grown.

Q. 43. Irrigation water is usually applied to the fields, when the available moisture in the root zone of the crop, gets depleted by :
(*a*) 0—10% (*b*) 10—25%
(*c*) 50—80% (*d*) 100%.

Q. 44. Frequency of irrigation is dependent upon the type of :
(*a*) soil and crop (*b*) soil and climate
(*c*) soil, crop, and climate (*d*) soil, crop, climate, and fertilizer.

Q. 45.* A soil has a field capacity of 25%, permanent wilting point of 15%, and specific weight of 14.7 kN/m^3. If the root zone depth of the grown crop is 90 cm, then its available moisture holding capacity is
(*a*) 10 cm (*b*) 13.5 cm
(*c*) 16.67 cm (*d*) 20 cm.

Q. 46. After how many days will you recommend supplying irrigation water to a crop having 0.8 m root zone depth, and grown in a soil having field capacity of 30%, permanent wilting = 15%, and density of soil = 1.5 gm/cc ? The consumptive use for the crop is 5 mm/day, and only 60% of available moisture is permitted to be availed.
(*a*) 9 days (*b*) 15 days
(*c*) 21 days (*d*) 40 days.

Q. 47.* If the irrigation efficiency at a 10 hectare field is 80%, and the conveyance losses from the canal outlet is 10%, then the volume of water required at the canal outlet, for supplying 10 cm water depth in the field, will be equal to;
(*a*) 10,390 kL (*b*) 7200 kL
(*c*) 13,900 kL (*d*) 1,39,000 kL.

Q. 48. The Δ (in m) for a crop, having quantity duty D (in ha/M. cum), and base period B (in days) is given by :
(*a*) $\Delta = \frac{8.64\,B}{D}$ (*b*) $\Delta = \frac{864\,B}{D}$
(*c*) $\Delta = \frac{100}{D}$ (*d*) $\Delta = \frac{10{,}000}{D}$.

*Hints are given at the end of the chapter, for solving starred questions.

Q. 49. The ratio of the water stored in the root zone during irrigation, to the water needed in the root zone prior to irrigation, is called :
(*a*) efficiency of water use
(*b*) efficiency of water storage
(*c*) efficiency of water application
(*d*) efficiency of water conveyance.

Q. 50. If the irrigation water applied to a field penetrates uniformly throughout, then the water distribution efficiency is :
(*a*) 1 (*b*) 0
(*c*) 0.5
(*d*) none of them, as more data is required to ascertain it.

Q. 51. The efficiency of water application does not depend upon :
(*a*) climatic conditions
(*b*) type of the soil
(*c*) method of application
(*d*) geometry of the conveyance system.

Q. 52. The efficiency of water conveyance does not depend upon :
(*a*) climatic conditions
(*b*) geometry of the conveyance system
(*c*) nature of the boundary of the conveyance system
(*d*) method of application of water.

Q. 53. Which of the following is not correctly matched?
(*a*) Rice—Kharif (*b*) Wheat—Rabi
(*c*) Barley—Kharif (*d*) Potatoe—Rabi.

CHAPTER 3

Q. 54. If the intensity of irrigation for Kharif is 45% and that for Rabi is 60%; then the annual intensity of irrigation, is :
(*a*) 60% (*b*) 100%
(*c*) 105% (*d*) none of them.

Q. 55.* If in a certain irrigation project, and in a given year, 72% and 56% of the culturable command remained unirrigated in Kharif and Rabi seasons, respectively; then the intensity of irrigation for that year and for that project, would be :
(*a*) 36% (*b*) 64%
(*c*) 72% (*d*) 128%.

Q. 56. The gross irrigated area cannot exceed the cultivable commanded area :
(*a*) Correct (*b*) Incorrect.

Q. 57.* The area, on which crops are grown in a particular season, is called :
(*a*) culturable commanded area (CCA)
(*b*) gross sown area (*c*) net sown area
(*d*) none of the above.

Q. 58.* The CCA for a particular State is 5 Mha; out of which, 4.5 Mha is being sown in Rabi season and 2.5 Mha in Kharif season. These areas are being irrigated to the extent of 90% and 80%, respectively. The annual intensity of irrigation for this State is :
(*a*) 80.7% (*b*) 85%
(*c*) 100% (*d*) 121%.

Q. 59. In plain areas, the irrigation canals are usually aligned along :
(*a*) ridge lines (*b*) contour lines
(*c*) valley lines (*d*) across the contours.

Q. 60. A ridge canal is also called a :
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of the above.

Q. 61. The canal, which may frequently encounter cross-drainage works, will be a:
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of these.

Q. 62. In a hilly district, where watershed line is very high as compared to the head-works, which type of alignment you would recommend for a proposed canal, if cross-drainage is to be avoided :
(*a*) canal along the ridge line
(*b*) canal along the contour line
(*c*) canal across the contour lines
(*d*) none of them.

Q. 63. The canal, which can irrigate only on one side, is a :
(*a*) watershed canal (*b*) contour canal
(*c*) side slope canal (*d*) none of them.

Q. 64. The capacity of an irrigation canal is usually controlled by :
(*a*) average kharif demand
(*b*) average rabi demand
(*c*) kor demand of rabi crops
(*d*) kor demand of kharif crops.

Q. 65. The optimum kor water depth for a kharif crop is 19 cm with an allowed kor water period of 3 weeks, the outlet discharge factor for this crop will be:
(*a*) 955 ha/cumec (*b*) 782 ha/cumec
(*c*) 860 ha/cumec (*d*) none of them.

Q. 66.* If the discharge required for different crops grown in a field is 0.4 cumecs, and the capacity factor and time factor are 0.8 and 0.5, the design discharge for the distributary will be :
(*a*) 0.80 cumecs (*b*) 0.16 cumecs
(*c*) 1 cumecs (*d*) 1.24 cumecs.

Q. 67.* An irrigation canal is to be designed to deliver 6.25 cumecs to meet the peak Rabi demand of a total of 5100 hectares of cropped area. The estimated canal losses are 25% of the head discharge. The duty on capacity of this canal will be :
(*a*) 816 ha/cumec (*b*) 1020 ha/cumec
(*c*) 653 ha/cumec (*d*) 612 ha/cumec.

Q. 68. 'Duty on capacity' is also called :
(*a*) outlet duty (*b*) capacity factor
(*c*) full supply coefficient (*d*) quantity duty.

Q. 69.* The 8000 hectares of gross commanded area of an irrigation project includes 20% of reserved forests, usar lands, roads, etc. The pastures and fallow lands are 10%. If the intensity of irrigation is 50%, the area to be irrigated is :
(*a*) 4000 hectares (*b*) 3200 hectares
(*c*) 2800 hectares (*d*) none of them.

Q. 70. The discharge carried by a minor distributary is usually less than :
(*a*) 0.5 cumecs (*b*) 1 cumecs
(*c*) 2.5 cumecs (*d*) none of them.

Q. 71. The discharge carried by a major distributary, is usually up to :
(*a*) 5 cumecs (*b*) 10 cumecs
(*c*) 30 cumecs (*d*) 50 cumecs.

Q. 72. Earthen irrigation canals, when aligned curvilinear in plan, must be on gentle curves. This adopted curve radius, should be :
(*a*) more for canal carrying more discharge
(*b*) less for canal carrying more discharge
(*c*) equal for all discharges.

Q. 73. The canal which is not supposed to do any irrigation is called :
(*a*) main canal (*b*) water course
(*c*) major distributary (*d*) minor distributary.

Q. 74. Unlined irrigation canals, when aligned on curvilinear routes in plan, will have to be pitched on :
(*a*) both sides (*b*) concave side only
(*c*) convex side only (*b*) none of the above.

CHAPTER 4

Q. 75. The bed of an alluvial channel along the flow will always be :
(*a*) flat (*b*) wavy
(*c*) duned and rippled (*d*) all of the above are possible.

Q. 76. The bed form, which is not expected in an alluvial channel with sediment motion, is :
(*a*) rippled (*b*) meandered
(*c*) duned (*d*) anti duned.

Q. 77. The bed of an alluvial channel carrying silted water at a high velocity is expected to be :
(*a*) rippled (*b*) duned
(*c*) flat (*d*) wavy.

Q. 78. The anti-dunes develop on beds of alluvial streams, when Froude number is:
(*a*) 0 (*b*) 0.5
(*c*) 1 (*d*) 1.2.

Q. 79.* The critical shear stress τ, at which incipient motion of sediment takes place, is proportional to :
(*a*) $\sqrt{d}$ (*b*) d
(*c*) d^2 where d is grain size (*UPSC Engg. Services*)

Q. 80.* In order to ensure that no scouring takes place in the bed of a channel of bed slope S, constructed in an incoherent alluvium of size 'd' cm, the flow velocity should be restricted to :
(*a*) $4.85 \cdot d^{1/2} \cdot S^{-1/6}$ (*b*) $4.85 \cdot d^{-1/2} \cdot S^{1/6}$
(*c*) $0.48\, d^{1/2} \cdot S^{1/6}$ (*d*) $0.48 \cdot d^{1/2} \cdot S^{-1/6}$.

Q. 81.* Manning's rugosity coefficient is proportional to :
(*a*) $\sqrt{d}$ (*b*) d
(*c*) $d^{1/6}$ (*d*) $d^{2/3}$
(*e*) $d^{3/2}$; where d is representative grain dia of bed surface.

Q. 82. The finer sediment which does not originate from the stream bed, but originates from the catchment area, is best called as the :

(*a*) bed load (*b*) suspended load
(*c*) wash load (*d*) none of these.

Q. 83. The force exerted by the flowing water on the sediment particles to cause their motion, is called :

(*a*) buoyant force (*b*) tractive force
(*c*) kinematic force (*d*) eddy force.

Q. 84. The minimum size of sediment that may remain stable in an alluvial channel, carrying discharge intensity q, with hydraulic radius R, and bottom slope S, is :

(*a*) $11\,RS$ (*b*) $1.35\left(\frac{q^2}{g}\right)^{1/2}$
(*c*) γRS (*d*) none of them.

Q. 85.* The tractive force method of designing stable channels in non-cohesive soils, is applicable only when the channels :

(*a*) carry sedimented water
(*b*) carry clear water
(*c*) carry sedimented water, but sediment concentration does not exceed 500 ppm or so
(*d*) carry any type of water.

Q. 86. In an earthen trapezoidal channel having side slopes of 1.5 H : 1 V, constructed in a non-cohesive soil, having D_{75} particle size of 'd' m, and angle $\phi = 37°$, no scouring will occur only when the hydraulic mean depth (R) of the channel is limited to :

(*a*) $\frac{d}{33\,S_0}$ (*b*) $\frac{d}{11\,S_0}$
(*c*) $\frac{d}{21\,S_0}$ (*d*) none of them.

Q. 87. If τ_c is the critical shear stress on a channel bed, then the critical shear on the side slopes (nH : IV) of this channel (τ_c') is given by :

(*a*) $\tau_c' = \tau_c \cdot \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}}$ (*b*) $\tau_c' = \tau_c \cdot \sqrt{1 + \frac{\sin^2\theta}{\sin^2\phi}}$
(*c*) $\tau_c' = \tau_c \cdot \sqrt{1 - \frac{\sin^2\phi}{\sin^2\theta}}$ (*d*) $\tau_c' = \tau_c \cdot \sqrt{1 + \frac{\sin^2\phi}{\sin^2\theta}}$

where ϕ = angle of repose of soil

$$\theta = \tan^{-1}\left(\frac{1}{n}\right)$$

Q. 88. The Lacey's and Kennedy's empirical silt theories for designing irrigation channels in Indian non-cohesive alluvial soils, are meant to ensure :

(*a*) no scouring in the channel
(*b*) no silting from out of the sedimented water entering the channel from the headworks
(*c*) both (*a*) and (*b*)
(*d*) neither (*a*) nor (*b*).

Q. 89.* The critical velocity $V_0 = 0.55 \cdot m \cdot y^{0.64}$, as suggested by Kennedy for design of trapezoidal irrigation channels, is :

(*a*) the maximum permissible velocity
(*b*) the minimum permissible velocity
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 90. Critical velocity Ratio for use in Kennedy's theory, is :

(*a*) less than 1 (*b*) more than 1
(*c*) equal to 1 (*d*) all of the above.

Q. 91. The Garret's diagrams are based on :

(*a*) Lacey's theory (*b*) Khosla's theory
(*c*) Bligh's theory (*d*) Kennedy's theory.

Q. 92. The critical velocity ratio was introduced in Kennedy's equation of critical velocity to take into account the effect of :

(*a*) channel cross-section (*b*) climatic conditions
(*c*) silt grade (*d*) roughness of bed.

Q. 93. The most important shape parameter in sediment analysis is :

(*a*) sphericity (*b*) shape factor
(*c*) roundness (*d*) form factor.

Q. 94. Lacey's silt factor is proportional to :

(*a*) $\sqrt{d}$ (*b*) d
(*c*) d^2 ; where d is the grain size.

(U.P.S.C.; Engg. Services)

Q. 95. In mechanics of sediment transport, the parameter $\varepsilon = \frac{\eta}{\rho}$ (where η is eddy viscosity and ρ is density) is called :

(*a*) kinematic viscosity (*b*) absolute viscosity
(*c*) eddy kinematic viscosity (*d*) none of the above.

Q. 96. In sediment analysis, the parameter which indicates the amount of wear the grain particle has undergone during relative movement of silt and water, is

(*a*) sphericity (*b*) shape factor
(*c*) roundness (*d*) form factor.

Q. 97. The wetted perimeter P of a stable channel is proportional to :

(*a*) Q (*b*) $\sqrt{Q}$
(*c*) Q^2 ; where Q is the discharge

(U.P.S.C.; Engg. Services)

Q. 98. The units of Shield's Entrainment function are :

(*a*) Newton/sq m (*b*) metre
(*c*) kg/m/sec (*d*) no dimensions.

Q. 99. The cross-section of natural silt transporting channels tend to have the shape of :

(*a*) trapezium (*b*) semi-ellipse
(*c*) semi-circle (*d*) none of them.

Q. 100. In Lacey's regime theory, the flow velocity is proportional to :

(*a*) $(Qf^2)^{1/2}$ (*b*) $\frac{Q}{f^2}$
(*c*) $(Qf^2)^{1/6}$

(U.P.S.C.; Engg. Services)

Q. 101. For all homogeneous fluids, the value of Von-Karman Universal Constant, is:

(*a*) 0 (*b*) 1

(*c*) 0.4 (*d*) 0.8.

Q. 102. The boundary shear stress, τ_0 in an open channel is given by :

(*a*) $\frac{\gamma h^2}{2}$ (*b*) γRS

(*c*) $\frac{1}{n} R^{2/3} S^{1/2}$ where h = depth

R = hydraulic radius

S = Slope

γ = Specific weight of fluid

n = Manning's roughness factor

(*U.P.S.C.; Engg. Services*)

Q. 103. Lacey's regime theory is not applicable to a canal in :

(*a*) true regime (*b*) initial regime

(*c*) final regime (*d*) none of them.

Q. 104.* When a channel is protected on the bed and sides with some protective material, and there is no possibility of change in its cross-section and longitudinal slope, then the channel is said to be in :

(*a*) initial regime (*b*) final regime

(*c*) permanent regime (*d*) true regime.

Q. 105.* Hydraulic depth is the ratio of :

(*a*) wetted area to wetted perimeter

(*b*) wetted area to top width

(*c*) wetted area to bottom width

(*d*) wetted perimeter to top width.

Q. 106. The unit of Chezy's coefficient C in Chezy's formula, is :

(*a*) $\sqrt{m/s}$ (*b*) m/s

(*c*) m (*d*) Nil.

Q. 107. For a most economical trapezoidal channel section :

(*a*) hydraulic mean radius equals the depth of flow

(*b*) hydraulic mean radius equals half the depth of flow

(*c*) bottom width is twice the depth of flow

(*d*) bottom width is half the depth of flow.

Q. 108.* The channels, whose boundaries are made of easily erodible loose soils, are called :

(*a*) mobile boundary channels

(*b*) rigid boundary channels

(*c*) non-alluvial channels

(*d*) none of the above.

Q. 109.* Lacey's scour depth for a stream, carrying a discharge of 3 cumecs per metre width and having a silt factor 1.2, is :

(*a*) 1.32 m (*b*) 2.64 m

(*c*) 3.96 m (*d*) 4.32 m. (*GATE 1992*)

Q. 110. Which one of the following is not the requirement of an ideal regime condition in Lacey's regime-theory ?

(*a*) the discharge in the channel is constant
(*b*) the channel flows through the same soil grade, as that of the sediment entering the channel from the headworks
(*c*) the sediment grade and its amount entering the channel is constant.
(*d*) the silt grade should consist of clay sized particles.
(*e*) the flow should be uniform.

Q. 111.* An irrigation canal proposed to be excavated in an incoherent alluvium of median size 0.4 mm, for carrying a discharge of 40 cumecs, should ideally have a bed slope of :

(*a*) 1 in 6000 (*b*) 1 in 5200
(*c*) 1 in 2000 (*d*) 1 in 1265.

Q. 112.* The silt factor 'f', as used by Lacey in his regime theory, was adjusted to be equal to 1 for the silt of Upper Bari-Doab alluvium. What was the median size of this alluvium ?

(*a*) 0.28 mm (*b*) 0.32 mm
(*c*) 0.40 mm (*d*) 0.5 cm.

Q. 113. Average sediment load in a regime canal of Lacey, is of the order of :

(*a*) 50 ppm (*b*) 500 ppm
(*c*) 5000 ppm (*d*) 10,000 ppm.

Q. 114.* The discharge carried by a regime canal of bed width 22 m and water depth 2.5 m, is about;

(*a*) 22 cumecs (*b*) 34 cumecs
(*c*) 84 cumecs (*d*) none of them.

Q. 115.* Two irrigation channels, A and B, are designed on Lacey's theory to carry the same discharge. The alluvium through which canal A has to pass, however, is coarser than that for canal B. In such a design, we expect :

(*a*) channel *A* to be deeper
(*b*) channel *B* to be deeper
(*c*) channel *A* to have larger wetted perimeter
(*d*) channel *B* to have larger cross-sectional area.

Q. 116. The major drawback of Laceys regime theory, as used for the design of irrigation canals, is that :

(*a*) it does not consider the quantum of sediment load, which is likely to flow into the canal
(*b*) It is applicable only to non-cohesive soils
(*c*) it is applicable only to alluvial areas of North India or Pakistan.
(*d*) none of the above.

Q. 117. "An alluvial river scours and silts its bed automatically, depending upon the variation in the flow discharge". Which theory helps us to know this important conclusion?

(*a*) Lacey's theory (*b*) Kennedy's theory
(*c*) Einstein's theory (*d*) none of them.

Q. 118. Stable channels for a given bed-load transport, can be best designed on the basis of

(*a*) Lacey's theory (*b*) Kennedy's theory
(*c*) Einstein's theory (*d*) all of the above.

Q. 119. In an irrigation canal, berms are provided at :

(*a*) NSL (*b*) FSL

(*c*) Bank level (*d*) none of the above.

Q. 120. Counter berms are provided in an irrigation canal :

(*a*) at the outer sides of the canal banks to allow movement of cattle

(*b*) at the inner sides of the canal banks at the ground level, to make bank line and bed line parallel

(*c*) at the outer sides of the canal banks to keep the saturation gradient line well within the bank section

(*d*) none of the above.

Q. 121. Berms are provided in an irrigation canal at :

(*a*) the outer edge of the canal banks to allow movement of cattle

(*b*) the inner sides of the canal bank at the ground level, to make inner bank line and bed line parallel

(*c*) the outer sides of the canal banks, to keep the phreatic line adequately covered by earth cover

(*d*) none of the above.

Q. 122.* An alluvial canal is excavated using side slopes of n_1 : 1 (H : V) in excavation and n_2 : 1 in filling. If the depth of cutting is d_1, and top of the bank is at a height of d_2 above the bed, then the berm width required to have the bed line and bank line parallel to each other, despite fluctuations in the ground level, is :

(*a*) $(n_2 - n_1)\, d_2$ (*b*) $(n_2 - n_1)\, d_1$

(*c*) $(n_2 - n_1)\,(d_2 - d_1)$ (*d*) $n_2 \cdot d_2$.

Q. 123.* An alluvial irrigation canal is constructed with 1 : 1 (H : V) side slopes in cutting and 2 : 1 in filling, and the water depth upto FSL is expected to be y. A berm width equal to y is left initially, which with the passage of time, and when channel attains regime conditions, is likely to become :

(*a*) 1.1 *y* (*b*) 1.25 *y*

(*c*) 1.5 *y* (*d*) 2*y*.

CHAPTER 5

Q. 124. A lined alluvial canal is best designed on the basis of :

(*a*) Lacey's formula (*b*) Kennedy's formula

(*c*) Manning's formula (*d*) Continuity equation.

Q. 125. Lining of irrigation channels :

(*a*) increases water logging (*b*) increases channel cross section

(*c*) increases command area (*d*) increases chances of breaching.

Q. 126.* An irrigation canal is lined along its 20 m perimeter @ Rs. 40 per m^2 of lining. The lining has resulted in net water saving @ 3 cumecs per M-m^2 of lined area, whereas each cumec of water increases the annual crop yield by Rs. 4 lakh. Assuming the life of lining as 40 years, and the saving in maintenance cost to be balanced by the interest on investment, the benefit cost ratio for the project is :

(*a*) 1.2 (*b*) 1.0

(*c*) 0.83 (*d*) none of them.

Q. 127. The transmission loss of valuable water in an ordinary alluvial irrigation canal in a North Indian State, in cumecs per million sq m of wetted area, is of the order of :

(*a*) 0.5—0.6 (*b*) 2.5—3.5

(*c*) 25—35 (*d*) 250—350.

Q. 128. A triangular lined canal section with corners rounded by a radius equal to the full supply depth of 4 m, is likely to have its hydraulic radius, as :

(*a*) 4 m

(*b*) 3 m

(*c*) 2 m

(*d*) cannot be ascertained, as side slopes are not given.

Q. 129.* A standard triangular lined channel section with 1.5 H : 1 V side slopes has a capacity of 26 cumecs and full supply depth of 2.5 m. The lining material for the channel would be safe for withstanding mean velocity of :

(*a*) 1.8 m/s (*b*) 2.0 m/s

(*c*) 2.2 m/s (*d*) none of them.

Q. 130. Safety ladders are provided in large irrigation canals, to :

(*a*) enable the fish to pass from one place to another

(*b*) enable the cattle to cross the canal

(*c*) enable the swimmers to get out of the canal

(*d*) provide safe exit to avoid accidental drowning.

Q. 131. The main advantage offered by Bentonite layer lining in an irrigation canal is that :

(*a*) seepage loss is reduced

(*b*) cross-sectional area of the channel is reduced

(*c*) cost of land acquired is reduced

(*d*) earth work excavation is reduced.

Q. 132. The free-board in lined canals is measured between :

(*a*) FSL and top of lining

(*b*) FSL and top of canal bank

(*c*) top of lining and top of canal bank

(*d*) none of them.

133. The minimum recommended free-board for lined canals carrying discharge of more than 10 cumecs is

(*a*) 0.3 m (*b*) 0.6 m

(*c*) 0.75 m (*d*) 1.2 m.

Q. 134. Under-drainage arrangements in canals are necessarily required in :

(*a*) unlined canals

(*b*) lined canals

(*c*) both (*a*) and (*b*)

(*d*) lined canals constructed on sandy soils alone.

Q. 135. When elaborate under-drainage arrangements can not be provided, and the soil strata is sandy, the preferred material for canal lining, would be :

(*a*) cement concrete lining

(*b*) soil cement lining

(*c*) brick lining

(*d*) shotcrete lining.

Q. 136. Steel reinforcement in concrete lining is justified, mainly because :
(*a*) it augments the structural strength of uncracked lining
(*b*) it enables the lining to take tensile loads
(*c*) it reduces the width of shrinkage cracks
(*d*) none of the above.

Q. 137. Which among the following is not true about Asphaltic concrete lining?
(*a*) it is fairly cheap
(*b*) it is flexible and readily confirms to subgrade
(*c*) it permits certain type of weed growth
(*d*) it decreases the rugosity coefficient to increase channel efficiency.

CHAPTER 6

Q. 138. A land is said to be water-logged, when :
(*a*) the land is necessarily submerged under standing water
(*b*) there is a flowing water over the land
(*c*) the pH value of the soil becomes as high as 8.5
(*d*) the soil pores in the root zone get saturated with water, either by the actual watertable or by its capillary fringe.

Q. 139. Water-logging of cropped land leads to reduced crop yields, due to :
(*a*) ill-aeration of root zone, causing lack of oxygen to plants
(*b*) growth of water-loving plants interfering with the sown crop
(*c*) surrounding of the root zone by the resultant saline water, which extracts the good water from plant roots by osmosis
(*d*) all of the above.

Q. 140. Pick up the incorrect statement from the following :
(*a*) intensive irrigation should be avoided in areas susceptible to water-logging
(*b*) extensive irrigation should be adopted in areas susceptible to water logging
(*c*) lift irrigation can help alleviate water-logging susceptibilities.
(*d*) none of the above.

Q. 141. Which one of the followings does not contribute to water logging?
(*a*) inadequate drainage
(*b*) seepage from unlined canals
(*c*) frequent flooding
(*d*) excessive tapping of ground water.

Q. 142. Which one of the followings, is not a remedial measure for water logging?
(*a*) good drainage for irrigated land
(*b*) conjunctive use of water in the basin
(*c*) lining of canals and water courses
(*d*) contour bunding.

Q. 143. Pick up the incorrect statement :
(*a*) moderately saline lands are represented by visible whiteish deposits of salts on the top and upper layers of soil
(*b*) $Na_2 CO_3$ is the most harmful salt for causing salinity
(*c*) Nacl is the least harmful salt for causing salinity
(*d*) a highly saline land is called an alkaline land, or a black alkali.
(*e*) none of the above.

Q. 144. Point out the incorrect statement, out of the following :
(*a*) salinity is caused by water-logging
(*b*) water logging is not caused by salinity
(*c*) salinity subsides, when once the water logging is removed
(*d*) none of the above.

Q. 145. Alkaline soils are best reclaimed by :
(*a*) leaching
(*b*) addition of gypsum to soil
(*c*) providing good drainage
(*d*) addition of gypsum to soil and leaching.

Q. 146. A recently reclaimed alkaline soil should preferably be sown with a salt resistant crop, like :
(*a*) wheat (*b*) cotton
(*c*) barseem (*d*) any of the above.

Q. 147.* A soil has an Exchangeable Sodium Percentage (ESP) of 16%, and its Electrical conductivity (EC) is 32 milli-mhos/cm. If the pH of this soil is 9.5, then the soil is classified as :
(*a*) alkaline soil (*b*) saline soil
(*c*) saline alkali soil (*d*) none of the above.

Q. 148. The soil becomes practically infertile when its pH value is about :
(*a*) 0 (*b*) 7
(*c*) 11 (*d*) none of them.

Q. 149. Which of the following crops can withstand the highest water table, which normally should be 0.7 to 2.5 m below the level of the cropped land?
(*a*) wheat (*b*) lucerine
(*c*) cotton (*d*) rice.

Q. 150. While growing crops in high water-table areas, open jointed drains, discharging into outlet drains, are sometimes laid below the cropped land to lower down the water-table and to remove the rain water effectively. These drains are called:
(*a*) tile drains (*b*) french drains
(*c*) gravity outlets (*d*) surface drains.

Q. 151. A tile drain is laid below a cropped land to remove excess irrigation water. The Drainage Coefficient of this drain, is usually expressed as;
(*a*) cm of water depth removed from the drainage area per day
(*b*) cum of water removed per second
(*c*) percentage of applied water, which is intercepted by this drain
(*d*) none of the above.

Q. 152. When a sub-surface open jointed drain is laid below a farm land to intercept excess irrigation water, then its capacity, depending upon soil type, should be of the order of :
(*a*) 5—10% of applied water
(*b*) 10—50% of applied water
(*c*) 50—80% of applied water
(*d*) none of them.

Q. 153. The method, which uses dead furrows on cropped farms for drainage of excess irrigation or rain water, is called :
(*a*) surface inlet (*b*) tile drainage
(*c*) bedding (*d*) french drain.

CHAPTER 7

250 Objective questions on this chapter are given in author's another book, titled *"Hydrology and Water Resources Engineering"*, which deals with the subject matter in greater details than given in this book. Students are advised to refer to that volume to be well versed with the subject on 'Hydrology', which in itself, is a full subject, other than from 'Irrigation'.

CHAPTER 8

Q. 154. Ghaggar river which appears and disappears in Rajasthan State of India, can be classified as a

(*a*) meandering river (*b*) deltaic river
(*c*) virgin river (*d*) none of the above.

Q. 155. The quantum of river discharge, which is large enough in magnitude as well as in frequency, so as to cause controlling effect on the river course and cross-section, is known as :

(*a*) design discharge (*b*) median discharge
(*c*) maximum flood discharge
(*d*) dominant discharge.

Q. 156. The ratio of dominant discharge for a river to the peak discharge for that river, is of the order of :

(*a*) 3/16 (*b*) 9/16
(*c*) 13/16 (*d*) 3/2.

Q. 157. The 'meander length' for an alluvial river is :

(*a*) the total channel length along its looped course
(*b*) the total channel length minus the direct straight length
(*c*) the axial length of one meander
(*d*) the looped length of one meander
(*e*) none of the above.

Q. 158. The 'meander belt' for an alluvial river is :

(*a*) the total river width between embankments
(*b*) the width between the outer edges of fully developed meander loop, measured perpendicular to river axis
(*c*) the same as meander width
(*d*) both (*b*) and (*c*).

Q. 159. For a meandering alluvial river, the ratio of its 'channel length' to 'direct axial length' is always :

(*a*) 1 (*b*) > 1
(*c*) < 1
(*d*) may be less or more than 1, depending upon the river.

Q. 160. In a meandering river, the ratio of 'arcual channel length' to 'direct axial length', is called :

(*a*) tortuosity (*b*) inverse of tortuosity
(*c*) cut off ratio (*d*) none of these.

Q. 161. Tortuosity in a meandering river, is :

(*a*) 1 (*b*) < 1
(*c*) > 1 (*d*) none of these.

Q. 162. A river reach having tortuosity of 1.2, can be said to have :

(*a*) 20% tortuosity of meander

(*b*) 80% tortuosity of meander

(*c*) 120% tortuosity of meander

(*d*) none of the above.

Q. 163. In an alluvial meandering river in flood plains, the meandering length (M_L) is x times the meandering width (M_B), where x is roughly equal to :

(*a*) 3 (*b*) $\frac{1}{3}$

(*c*) 1 (*d*) $\frac{1}{3}$ to 3.

Q. 164. For a meandering alluvial river in flood plain, the meander length is about:

(*a*) 6 W (*b*) 18 W

(*c*) 17 W (*d*) W.

where W is the normal river width

Q. 165. For a meandering alluvial river in flood plain, the meander width is about:

(*a*) 0.5 W (*b*) W

(*c*) 6 W (*d*) 17 W.

where W is the normal river width

Q. 166.* An alluvial river has a dominant discharge of 1600 m^3/s and a bed slope of 1 in 5000. The approximate value of meander belt for this river, would be :

(*a*) 0.6 km (*b*) 6 km

(*c*) 60 km (*d*) none of these.

Q. 167. An alluvial meandering river is likely to carry a peak discharge of 6000 cumecs. The extent of meandering belt in this river, estimated in km per km of meandering length, roughly equals.

(*a*) $\frac{1}{3}$ (*b*) 3

(*c*) 9 (*d*) 27.

Q. 168. Meander ratio in an alluvial meandering river is given by :

(*a*) $\frac{\text{meander length}}{\text{meander width}}$ (*b*) $\frac{\text{meander width}}{\text{meander length}}$

(*c*) $\frac{\text{meander width}}{\text{meander length}} \times 100$ (*d*) none of them.

Q. 169.* The meander belt for an alluvial stream is estimated to be 3 km. The peak discharge in this river will roughly be of the order of :

(*a*) 4000 cumecs (*b*) 700 cumecs

(*c*) 1200 cumecs (*d*) 1600 cumecs.

Q. 170. In a meandering river reach, the deepest river portions will be available at:

(*a*) the crossings

(*b*) the inner edges of meander loops

(*c*) the outer edges of meander loops

(*d*) none of them.

Q. 171. Sinuosity of a meandering river is :

(*a*) the same as tortuosity (*b*) inverse of tortuosity

(*c*) log of tortuosity (*d*) none of them.

Q. 172. Pinpoint the incorrect statement :

(*a*) guide-bunds are constructed to guide the river flow through a smaller width

(*b*) guide bunds are generally extended downstream of the hydraulic structure, in a length of about twice their upstream length

(*c*) guide banks prevent the out-flanking of the hydraulic structure by the changing river course

(*d*) aggrading rivers can be best controlled by soil conservation measures, including construction of check dams on their tributaries

(*e*) degrading rivers can be best controlled by building of cross-bars, weirs, etc., prior to other river training works.

Q. 173. Aggrading rivers are :

(*a*) silting rivers (*b*) scouring rivers

(*c*) rivers in regime (*d*) meandering rivers.

Q. 174. The river reach upstream of a newly built dam may behave, as :

(*a*) aggrading (*b*) degrading

(*c*) virgin (*d*) none of them.

Q. 175. A barrage with 600 m length is designed across an alluvial river. The U/S and D/S lengths of the proposed guide banks of this barrage, will be about:

(*a*) 750 m and 1500 m (*b*) 750 m and 750 m

(*c*) 750 m and 150 m (*d*) 750 m and 300 m

Q. 176.* A pair of guide banks are to be constructed for an alluvial river at 400 m apart at a bridge site. The design discharge of the river is 5000 cumecs at 5 m water depth of HFL above the average bed level of the river. These guide banks need to be stone pitched on water side, with apron laid from the toe in the river bed. The average estimated depth and width of this apron, respectively, will be :

(*a*) 2 m and 10 m (*b*) 1.5 m and 5 m

(*c*) 2 m and 20 m (*d*) 3 m and 20 m.

Q. 177. The repelling groynes which are largely constructed projecting from river embankments, as anti-erosion works, are :

(*a*) pointing upstream (*b*) pointing downstream

(*c*) perpendicular to the bank (*d*) none of these.

Q. 178.* If a river in an alluvial plain has a dominant discharge of 1600 cumecs, the waterway for this bridge, in the absence of model testing, can be safely adopted, as equal to :

(*a*) 190 m (*b*) 228 m

(*c*) 300 m (*d*) 480 m.

Q. 179. The upstream angle of inclination of a repelling groyne with normal to the bank line, is of the order of :

(*a*) 5 to 10° (*b*) 10 to 30°

(*c*) 30 to 50° (*d*) 70 to 90°.

Q. 180. Which one among the following is not the real cause of an alluvial river following a serpentine course?

(*a*) variation of discharge

(*b*) availability of steeper land gradient than what is required for equilibrium.

(*c*) availability of inadequate land gradient than what is required for equilibrium

(*d*) none of these.

Q. 181. An alluvial river increases its length by meandering due to;
(*a*) variation of discharge (*b*) variation in land topography
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 182. The factor which is not primarily responsible for meander in an alluvial river, is;
(*a*) variation of discharge (*b*) excess silt load
(*c*) deficient silt load (*d*) none of them.

Q. 183. The secondary factor, which is responsible for meandering in an alluvial river, is :
(*a*) inadequate land gradient (*b*) deficient silt load
(*c*) constant discharge (*d*) none of them.

Q. 184. Point out the incorrect statement in relation to the behaviour of a meandering alluvial river :
(*a*) Primary cause for meandering is the silting action due to excess sediment at the available discharge and surface slope
(*b*) Secondary cause for meandering is the scouring action due to deficiency of sediment at the available discharge and surface slope
(*c*) Meandering tendency is more in an alluvial river during floods, because of high variation in discharge
(*d*) if there is no variation of discharge in an alluvial river, there will be no meandering
(*e*) none of them.

Q. 185. Out of the following choices given below, choose impermeable spur(s) :
(*a*) an earthen spur protected by stone apron
(*b*) an earthen spur, unprotected by stone apron
(*c*) a balli spur (*d*) a tree groyne
(*e*) (*a*) and (*b*) both (*f*) (*c*) and (*d*) both.

Q. 186. Permeable spurs are best suitable for rivers, which :
(*a*) carry heavy suspended load
(*b*) carry large bed load, but light suspended load
(*c*) need permanent protection to dikes
(*d*) need attracting the river current, for providing deeper channel
(*e*) flow in upper hilly reaches.

Q. 187. Denehey's spur is :
(*a*) a hockey-shaped earthen spur
(*b*) a *T*-shaped stone spur, as used in Australia
(*c*) a *T*-shaped earthen spur, as used in India
(*d*) a type of a balli spur, especially developed for Indian rivers.

Q. 188. Which one of the following effects can not be attributed to have been caused by the construction of dikes along a river coarse?
(*a*) faster travel of a flood wave
(*b*) higher flood levels along the river
(*c*) increased peak discharges all along downstream points
(*d*) decrease in surface slope of the river above the leveed reach
(*e*) increased bed levels in the river
(*f*) reduction in meander belt, causing steeper hydraulic gradients.

Q. 189. Which one of the following effects produced by a Cut off in an alluvial river, is not an advantage to navigation :

(*a*) shortened route and elimination of sharp bends
(*b*) shortened travel time, particularly at low and moderate river stages
(*c*) increased water depth at low river stages
(*d*) lowering of the flood stages and flood periods.

CHAPTER 9

Q. 190. Barrages constructed across alluvial rivers help in :

(*a*) controlling floods (*b*) restoring river regime
(*c*) ensuring monsoon storage (*d*) all of them.

Q. 191. Which one among the following is a correct choice in relation to a weir?

(*a*) it is helpful in diverting excess water to a river from a canal
(*b*) it does not cause any heading up of water on its upstream side
(*c*) it stores water by raised counter-balanced gates
(*d*) it increases the chances of floods in the upstream areas
(*e*) none of them.

Q. 192. If two canals are taken off from both the flanks of a river at the site of a diversion headwork, then the number of undersluices and divide walls, will respectively, be :

(*a*) 1 and 1 (*b*) 1 and 2
(*c*) 2 and 1 (*d*) 2 and 2.

Q. 193. Pinpoint the incorrect statement :

(*a*) the old Okhla barrage near Delhi has recently been replaced by a new weir
(*b*) the cost of a barrage is usually higher than that of a weir
(*c*) a weir can pond up additional water on its upstream side, by using shutters on its top
(*d*) a barrage does not increase the susceptibility of flooding in upstream areas
(*e*) none of them.

Q. 194. In a barrage project, a divide wall is provided to :

(*a*) separate the lower crest 'undersluice side' from the higher crest 'weir side'
(*b*) separate the higher crest 'undersluice side' from the lower crest 'weir side'
(*c*) keep the cross-currents away from the barrage body
(*d*) serve none of the above purposes.

Q. 195. A canal headworks has nothing to do with a :

(*a*) weir (*b*) guide bank
(*c*) head regulator (*d*) safety ladder.

Q. 196. In a diversion headworks project, the canal head regulator is usually aligned:

(*a*) parallel to the barrage axis
(*b*) perpendicular to the divide wall
(*c*) parallel to the divide wall
(*d*) none of them.

Q. 197. The choice among the following, which does not control the discharging capacity to be provided for the undersluices, is :
(*a*) that it should be at least double the canal discharge
(*b*) that it should be about 10 to 15% of maximum flood discharge
(*c*) that it should be able to pass winter freshets and low floods.
(*d*) that it should be equal to the dominant discharge of the river
(*e*) none of them.

Q. 198. Anadromous fish often move large distances in rivers in India :
(*a*) to upstream only
(*b*) to downstream only
(*c*) to downstream in winter, and upstream in summer
(*d*) to upstream in winter, and downstream in summer.

Q. 199. A fish ladder is provided in a canal project :
(*a*) to catch the fish for commercial development
(*b*) to enable the fish to move freely in the river
(*c*) to serve the same purpose as a canal ladder.
(*d*) (*b*) and (*c*) both.

Q. 200. Head sluices are the gate controlled openings, in :
(*a*) the entire length of the barrage
(*b*) the under-sluice length of the barrage
(*c*) the regulator of the main off taking canal
(*d*) none of them.

Q. 201. Silt excluders are constructed :
(*a*) on the river bed downstream of the head regulator
(*b*) on the river bed upstream of the head regulator
(*c*) on the canal bed downstream of the canal head regulator
(*d*) none of these.

Q. 202. The silt exclusion device, constructed on the bed of the main canal, taking off from a headwork, is called :
(*a*) silt excluder (*b*) silt ejector
(*e*) both (*a*) and (*b*) (*d*) none of them.

Q. 203. The tunnel openings provided infront of a canal head regulator at a Diversion headworks :
(*a*) discharge sedimented water into the canal
(*b*) discharge sediment load into the under-sluices, from where it ejects out to the downstream river
(*c*) discharge clear water into the canal
(*d*) none of these.

Q. 204. Canal headworks in the upper rocky reaches of the rivers, are uncommon, because :
(*a*) more drops are required on the resulting canal system.
(*b*) costly head regulator is required
(*c*) more cross-drainage works are required on the resulting canal network
(*d*) all of the above.

CHAPTER 10

Q. 205. If y_1 and y_2 are water depths upstream and downstream of a hydraulic jump respectively, the loss of energy due to hydraulic jump is :

(*a*) $(y_2 - y_1)^4/4y_1 y_2$ (*b*) $(y_2 - y_1)^3/4y_1y_2$

(*c*) $(y_2 - y_1)^2/4y_1y_1$ (*d*) $(y_2 - y_1)/4y_1y_2$. (GATE 1990)

Q. 206. A hydraulic jump involves;

(*a*) subcritical flow (*b*) super critical flow

(*c*) critical flow (*d*) all of the above.

Q. 207. A hydraulic jump ensures :

(*a*) change of subcritical flow to super critical flow

(*b*) change of super critical flow to subcritical flow

(*c*) change of subcritical flow to critical flow

(*d*) change of super critical flow to critical flow.

Q. 208. The phenomenon of hydraulic jump leads to :

(*a*) evolution of energy

(*b*) dissipation of energy

(*c*) sometimes (*a*) and sometimes (*b*)

(*d*) none of these.

Q. 209. If a vertical line is drawn on the specific energy curve, corresponding to a particular value of specific energy, then the two depth values are obtained which are known as :

(*a*) initial and sequent depths (*b*) alternate depths

(*c*) conjugate depths (*d*) all of these.

Q. 210. The sequent depth is always :

(*a*) less than the alternate subcritical depth

(*b*) more than the alternate subcritical depth

(*c*) equal to the alternate subcritical depth

(*d*) none of the above.

Q. 211. The formula connecting initial depth (y_1) with sequent depth (y_2) in a hydraulic jump, is given as :

(*a*) $y_2 = \frac{-y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$ (*b*) $y_2 = \frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$

(*c*) $y_2 = \frac{-y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{q^2}{gy_1}}$ (*d*) $y_2 = \frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{q^2}{gy_1}}$

where q is the discharge intensity in the channel at the section of the hydraulic jump.

Q. 212. The hydraulic jump that develops usually in barrages and canal head regulators, is of the type :

(*a*) strong jump (*b*) steady jump

(*c*) oscillating and weak jump (*d*) undular jump.

Q. 213. The formation of a hydraulic jump on a sloping glacis, as compared to that on a horizontal floor, is always :

(*a*) more definite and more efficient

(*b*) more definite and less efficient

(*c*) less definite and more efficient

(*d*) less definite and less efficient.

CHAPTER 11

Q. 214. The safety of a hydraulic structure founded on pervious foundation can be ensured :

(*a*) by providing sufficient length of its concrete floor
(*b*) by providing sufficient depth of its concrete floor
(*c*) by providing a downstream cutoff of some reasonable depth
(*d*) all of the above.

Q. 215. Point out the correct meaning of 'piping', as applied to the design of barrages:

(*a*) it refers to a network of pipes laid below the hydraulic structure to remove the seeping water
(*b*) it refers to the process of undermining of foundation and creation of hollows therein
(*c*) it refers to the uplift force caused by seeping water on the floor of the hydraulic structure
(*d*) none of the above.

Q. 216. While designing the downstream floor thickness of a weir, the ordinate of the uplift pressure at a point is 2.8 m. If the relative density of concrete is taken to be 2.4, then the minimum thickness of the floor to be provided for resisting uplift pressure, without accounting safety factor, is :

(*a*) 1.16 m (*b*) 2 m
(*c*) 0.8 m (*d*) none of them.

Q. 217.* While designing the floor thickness of a weir in its upstream length, the average head causing uplift is 2.8 m. The provided floor thickness here, with suitable factor of safety, could best be :

(*a*) 2 m (*b*) 2.5 m
(*c*) 0.8 m (*d*) 4.5 m.

Q. 218. The critical exit gradient suggested in Khosla's theory of design of weirs and barrages, is :

(*a*) less for more porous soils (*b*) more for more porous soils
(*c*) equal for all kinds of soils (*d*) none of them.

Q. 219.* Khosla's safe exit gradient for design of weirs will be the lowest for the soil type :

(*a*) fine sand (*b*) coarse sand
(*c*) shingle and gravels (*d*) none of them.

Q. 220. Point out the incorrect statement in relation to the design of weirs and barrages :

(*a*) Bligh's safe hydraulic gradient is the same as the Khosla's safe exit gradient
(*b*) the first streamline below a barrage section follows the bottom profile of the section
(*c*) equipotential lines are the lines joining the points of equal residual seepage head
(*d*) none of them.

Q. 221. Which one of the following factors, is not to be considered in the design of major hydraulic structures founded on an alluvial North Indian River?

(*a*) uplift caused by steady seepage
(*b*) piping caused by steady seepage
(*c*) uplift caused by high flood flow in the jump trough
(*d*) none of the above.

Q. 222. Bligh's theory, as applied to the design of weirs and barrages on permeable foundations, account for :

(*a*) hydrostatic forces only
(*b*) hydrodynamic forces only
(*c*) both (*a*) and (*b*)
(*d*) none of them.

Q. 223. Retrogression is :

(*a*) the back water effect of a weir
(*b*) the raising of the river bed upstream of the weir, during initial years of its construction
(*c*) the lowering of the river bed downstream of the weir, during initial years of its construction
(*d*) none of them.

Q. 224. According to Khosla's theory of independent variables for seepage below a hydraulic structure, the exit gradient, in the absence of a downstream sheet pile, is :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) none of them.

Q. 225. The value of Khosla's critical exit gradient for usually met alluvial sandy soils of our country, is about :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) $\frac{1}{4}$ to $\frac{1}{6}$.

Q. 226. The value of Khosla's safe exit gradient for usually met alluvial river soils of our country is :

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) $\frac{1}{4}$ to $\frac{1}{6}$.

Q. 227. Critical exit gradient, as enunciated in Khosla's theory of design of weirs and barrages on pervious foundations, is :

(*a*) the left out pressure in the seeping water at the downstream end point, where it emerges out on the river bed
(*b*) the rate of loss of pressure of the seeping water at the downstream emerging point, which is just enough to lift the soil grains at that point
(*c*) the actual pressure gradient of the seepage, at the down-stream emerging point
(*d*) none of the above.

Q. 228. Safe exit gradient, as enunciated by Khosla's theory of design of weirs and barrages, is :

(*a*) the actual seepage gradient available at the downstream exit point
(*b*) the seepage gradient at the downstream exit point, equalling the submerged weight of a unit volume of soil there
(*c*) about $\frac{1}{4}$ to $\frac{1}{6}$ times the gradient which is just able to lift the soil grain at the downstream exit point
(*d*) none of the above.

Q. 229. The critical exit gradient, as applicable to barrages, is expressed by :

(*a*) $w(1-n)(S_s-1)$,

where w = unit weight of water
n = porosity of soil
S = specific gravity of soil grains

(*d*) $\frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}}$

where H, d, and λ have their usual meaning

(*c*) $\frac{\text{Safe exit gradient}}{\text{Factor of safety}}$

(*d*) none of the above.

Q. 230. The minimum thickness(t) of the downstream floor, as required in the design of weirs, can be expressed by the equation :

(*a*) $\frac{h}{G+1}$ (*b*) $\frac{h}{G-1}$

(*c*) $\frac{h-t}{G-1}$ (*d*) $1.33\left(\frac{h}{G-1}\right)$

where h = ordinate of hydraulic gradient line above the top of the floor
G = Specific gravity of floor material.

Q. 231. The back water effect of a weir is best called :

(*a*) retrogression (*b*) afflux
(*c*) back water curve (*d*) none of them.

Q. 232. A breast wall is usually provided :

(*a*) in the weir section
(*b*) in the under-sluice section
(*c*) in the main canal section
(*d*) in the head regulator section.

Q. 233. Just downstream of the pucca concrete floor of a barrage section, an inverted filter covered by c.c. blocks is laid in a length of about 1.5 to 2 D, where D is :

(*a*) the Lacey's normal scour depth
(*b*) 1.5 times the Lacey's normal scour depth
(*c*) 1.5 times the Lacey's scour depth minus the downstream water depth
(*d*) none of the above.

Q. 234. Just downstream of the concrete floor of a weir section, an inverted filter covered by c.c. blocks is laid. This filter should be such, as :

(*a*) to let out the residual seepage
(*b*) not to allow the soil particles to escape
(*c*) not to get clogged in itself
(*d*) all of the above.

Q. 235. The safety against any possible scour, on upstream or downstream side of the pucca floor of a hydraulic structure, is usually ensured by laying :

(*a*) inverted filter (*b*) toewall
(*c*) rock toe (*d*) stone apron.

Q. 236. For the head regulator, the most severe condition of uplift pressure on the floor occurs when :

(*a*) the flow in the river is at flood level and canal is running at full supply depth

(*b*) the canal runs dry and the river flow is at high flood level

(*c*) the canal runs at full supply depth and the river flow is at pond level

(*d*) the canal runs dry and the river flow is at pond level. (*GATE 1992*)

CHAPTER 12

Q. 237. Canal drops are required to :

(*a*) dissipate excess energy

(*b*) dissipate inadequate land slope

(*c*) dissipate excess land slope

(*d*) none of the above.

Q. 238. A trapezoidal notch fall can maintain normal water depth in the upstream channel :

(*a*) at any one given value of the design discharge

(*b*) at all the discharges

(*c*) at any two values of the design discharges

(*d*) at no discharge at all.

Q. 239. The depth-discharge relationship of the upstream canal remains practically unaffected by the introduction of a fall of the type :

(*a*) Ogee fall

(*b*) Sarda type vertical fall

(*c*) Trapezoidal notch fall

(*d*) none of the above.

Q. 240. For low to moderate discharges of the order of 10-15 cumecs and fall height of 1 to 1.5 m, cheaper Sarda type canal falls can be recommended :

(*a*) universally, without any if and buts

(*b*) if the canal runs with highly variable discharge

(*c*) when the canal fall is not to be used as a meter and discharge is fairly constant

(*d*) when the canal fall is not to be used as a meter, irrespective of variation in discharge.

Q. 241. The type of canal fall, which can be universally used without any if and buts for moderate discharges of the order of 40-60 cumecs and low fall heights of 1 to 1.5 m, is :

(*a*) vertical drop fall (*b*) ogee fall

(*c*) glacis fall (*d*) baffle fall.

Q. 242. The type of fall, which you may recommend for very high drops and very low discharges, is :

(*a*) Sarda type fall (*b*) Siphon well drop

(*c*) Straight glacis fall (*d*) Inglis fall.

Q. 243. The canal fall, involving parabolic glacis, is called :

(*a*) Straight glacis fall (*b*) Glacis fall

(*c*) Inglis fall (*d*) Montague fall.

Q. 244. An Inglis fall, also called a Baffle fall, can be recommended for all discharges, provided :
(*a*) the fall is more than 1.5 m
(*b*) the fall is undrowned
(*c*) the fall is either flumed or unflumed
(*d*) all of the above.

Q. 245. The energy dissipation in a Sarda type canal drop is caused by :
(*a*) hydraulic jump (*b*) friction blocks
(*c*) water pool (*d*) baffle wall.

Q. 246. The best energy dissipation on the down-stream side of a canal drop, is caused in :
(*a*) Sarda type fall (*b*) Glacis fall
(*c*) Ogee fall (*d*) Montague fall.

Q. 247. A baffle wall and a water pool to dissipate energy, was introduced as an improvement, in a :
(*a*) Glacis fall (*b*) Montague fall
(*c*) Inglis fall (*d*) none of the above.

Q. 248. A meter fall of 1.8 m height is to be constructed on a canal carrying a discharge of 50 cumecs. The type of fall that you may recommend will be :
(*a*) Glacis fall (*b*) Montague fall
(*c*) Inglis fall (*d*) none of the above.

Q. 249. The energy dissipation in an Inglis fall is caused by :
(*a*) a pool of water (*b*) a hydraulic jump
(*c*) neither (*a*) nor (*b*) (*d*) both (*a*) and (*b*).

Q. 250. The length of the water cistern to be provided in a Sarda type fall, is :
(*a*) $5\,(y_2 - y_1)$ (*b*) $5\sqrt{H \cdot H_L}$
(*c*) $\frac{1}{4}(H \cdot H_L)^{2/3}$ (*d*) $1.84 \cdot H^{3/2}$.

where y_1, y_2, H, H_L have their usual meaning.

Q. 251. The concrete glacis of a glacis fall, usually slopes H : V, as :
(*a*) 2 : 1 (*b*) 1 : 2
(*c*) 1 : 5 (*d*) 5 : 1.

Q. 252. In a glacis fall, the hydraulic jump forms on :
(*a*) the horizontal floor (*b*) the sloping glacis
(*c*) either (*a*) or (*b*) (*d*) none of the above.

Q. 253. Friction blocks on the downstream side of the pucca canal fall, can be used to dissipate the left out energy in the falling water, in case of :
(*a*) a glacis fall (*b*) a vertical drop fall
(*c*) a baffle fall (*d*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 254. A trapezoidal crest in a Sarda type canal fall is preferred and used in comparison to a rectangular crest, when :
(*a*) the discharge is less than 14 cumecs
(*b*) the discharge is equal to or more than 14 cumecs
(*c*) the discharge is less than 30 cumecs
(*d*) the discharge is equal to or more than 30 cumecs.

Q. 255. A cylinder fall, popularly known as syphon well drop, is suitable and economical for :

(*a*) low discharges and low drops
(*b*) low discharges and high drops
(*c*) high discharges and low drops
(*d*) high discharges and high drops.

Q. 256. In a baffle fall, the hydraulic jump forms, on :

(*a*) the sloping glacis (*b*) the horizontal baffle platform
(*c*) either (*a*) or (*b*) (*d*) none of the above.

Q. 257. A baffle wall, as a sort of a low weir, is constructed in a baffle type canal fall at the end of the baffle platform and before the water cistern, inorder to head up water to its upstream side to :

(*a*) ensure formation of hydraulic jump to dissipate energy
(*b*) withstand actual impact of the high velocity jet to dissipate energy
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

CHAPTER-13

Q. 258. The canal regulator, which is constructed at a diversion headworks, is called a :

(*a*) cross regulator (*b*) distributary head regulator
(*c*) canal module (*d*) none of the above.

Q. 259. The gated regulator, which is constructed in the parent canal near the site of an offtaking canal, is called a :

(*a*) canal head regulator (*b*) distributary head regulator
(*c*) cross regulator (*d*) none of the above.

Q. 260. A cross regulator helps in

(*a*) increasing supply in the parent channel downstream
(*b*) increasing supply in the offtaking channel
(*c*) increasing water depth in the parent canal, upstream
(*d*) both (*b*) and (*c*)
(*e*) all (*a*), (*b*) and (*c*) above.

Q. 261. Point out the choice among the following, which is not a function of a distributary head regulator :

(*a*) it serves as a meter for measuring discharge in the offtaking canal
(*b*) it serves to control silt entry into the offtaking canal
(*c*) it helps in controlling and regulating supplies in the entire downstream canal network.
(*d*) it helps in controlling supplies in the offtaking canal.
(*e*) none of the above.

Q. 262. Khosla's theory of independent variables is used in the design of :

(*a*) weirs and barrages (*b*) cross regulators and head regulators
(*c*) modules (*d*) both (*a*) and (*b*) above
(*e*) all (*a*), (*b*) and (*c*) above.

Q. 263. The channel constructed to bypass the excess water entering a canal, is called a :

(*a*) canal module (*b*) canal siphon
(*c*) canal escape (*d*) canal regulator.

Q. 264. The crest level of a distributary head regulator is usually kept :
(*a*) the same as the upstream bed level of the parent canal
(*b*) the same as the crest level of the cross-regulator at the site.
(*c*) 0.3 to 1.0 m higher than the upstream bed level of the parent canal
(*d*) none of the above.

Q. 265. The regulator constructed on a major distributary near its junction with a minor distributary, will be called a :
(*a*) distributary head regulator
(*b*) cross-regulator
(*c*) regulating head
(*d*) none of the above.

Q. 266. The arrangement made in a canal network, which acts as its safety valve, is:
(*a*) cattle crossing (*b*) canal ladder
(*c*) canal escape (*d*) canal module.

Q. 267. The arrangement provided in a canal to help the cattle safely cross the canal, is called a :
(*a*) canal ladder (*b*) canal crossing
(*c*) cattle crossing (*d*) all of the above.

Q. 268. Point out the correct statement :
(*a*) bed bars are constructed in canals at the toe of the canal lining, to prevent slippage of lining.
(*b*) cattle crossings, besides their usual function of helping the cattle to cross the canal, also provide safety to those who are swept away by currents.
(*c*) a canal escape helps in supplying irrigation water to the downstream water courses
(*d*) none of the above.

Q. 269. Canal modules help in :
(*a*) modulating and varying the canal discharge
(*b*) releasing desired quantity of water into the water courses
(*c*) releasing desired quantity of water into the minors
(*d*) all of the above.

Q. 270. Canal outlets are also called :
(*a*) canal escapes (*b*) canal modules
(*c*) canal offtakes (*d*) canal openings.

Q. 271. The type of irrigation module, which makes equitable distribution of water more difficult, is a :
(*a*) non-modular outlet (*b*) semi-module
(*c*) rigid module (*d*) none of them.

Q. 272. A good irrigation module is the one, which :
(*a*) draws heavy silt from the canal
(*b*) draws clear water from the canal
(*c*) draws fair share of silt from the canal
(*d*) none of the above.

Q. 273. If the rate of change of discharge from an irrigation outlet is equal to the rate of change of discharge in the distributary, then the outlet is called :
(*a*) flexible (*b*) proportional
(*c*) sensitive (*d*) none of these.

Q. 274. The rate of change of discharge through an irrigation outlet becomes equal to the rate of change of water depth in the channel, when its :
(*a*) flexibility is 1 (*b*) sensitivity is 1
(*c*) setting is 1 (*d*) sensitivity is zero.

Q. 275. If the sensitivity of an irrigation module is $\frac{1}{2}$, then 50% variation in canal water depth will cause x% variation in outlet discharge, where x is :
(*a*) 100% (*b*) 50%
(*c*) 25% (*d*) 12.5%.

Q. 276. A fully modular canal outlet has its :
(*a*) sensitivity = 1, and flexibility = 1
(*b*) sensitivity = 1, and flexibility = 0
(*c*) sensitivity = 0, and flexibility = 1
(*d*) sensitivity = 0, and flexibility = 0.

Q. 277. An irrigation outlet is said to be proportional, when its :
(*a*) sensitivity = 1 (*b*) flexibility = 1
(*c*) setting = 1 (*d*) all of the above.

Q. 278. If the percentage rise/fall in the outlet discharge of a canal distributary is more than the percentage rise/fall of discharge in the distributary, then the outlet is :
(*a*) super sensitive (*b*) sub sensitive
(*c*) hyper proportional (*d*) sub proportional.

Q. 279. An irrigation, outlet fixed at a height of 0.4 y above the canal DBL, where y is the full supply depth in the canal, will have its setting equal to :
(*a*) 0.4 (*b*) 0.6
(*c*) 0.2 (*d*) none of these.

Q. 280. An irrigation outlet is said to be proportional, when its :
(*a*) setting = outlet index/channel index
(*b*) setting = channel index/outlet index
(*c*) setting = channel index × outlet index
(*d*) none of the above.

Q. 281. For a wide trapezoidal channel, the channel index is :
(*a*) 2/3 (*b*) 5/3
(*c*) 1/3 (*d*) none of them.

Q. 282. A free pipe outlet is a :
(*a*) rigid module (*b*) flexible module
(*c*) non-modular module (*d*) all of the above.

Q. 283. The type of irrigation module, which is usually considered as the best in our country, is :
(*a*) Adjustable orifice semi module
(*b*) Punjab open flume outlet
(*c*) Khanna's rigid module
(*d*) Kennedy's gauge module.

Q. 284. The only irrigation semi module, through which the discharge is not proportional to $\sqrt{H}$, where H is the head causing flow through the module, is :
(*a*) kennedy's gauge outlet (*b*) open flume outlet
(*c*) free pipe outlet (*d*) adjustable orifice semi module.

CHAPTER 14

Q. 285. The hydraulic structure which is constructed at the junction of a canal and a drain to dispose of drainage without disturbing canal supplies, is known as a;

(*a*) cattle crossing (*b*) canal module
(*c*) canal regulator (*d*) canal crossing.

Q. 286. The type of canal alignment, which involves maximum cross-drainage works, is a :

(*a*) ridge canal (*b*) contour canal
(*c*) side slope canal (*d*) all of the above.

Q. 287. The irrigation structure which is constructed at the crossing of a canal and a drain, to avoid the mixing of their respective discharges, is :

(*a*) an aqueduct (*b*) a super passage
(*c*) a level crossing (*b*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 288. When an irrigation canal is taken over a drainage channel, the crossing is called :

(*a*) an aqueduct (*b*) a super passage
(*c*) a level crossing (*d*) none of them.

Q. 289. An irrigation canal, freely flowing under a drainage channel, is specifically called a :

(*a*) canal junction (*b*) canal crossing
(*c*) canal siphon (*d*) super passage.

Q. 290. An irrigation canal flowing freely above a drainage, which in turn is flowing under pressure, is specifically, called a :

(*a*) canal siphon (*b*) canal aqueduct
(*c*) siphon aqueduct (*d*) super passage.

Q. 291. The crossing arrangement, preferably made at the junction of a huge canal and a river stream carrying short lived-high flood discharge at almost equal bed levels, is a :

(*a*) super passage (*b*) aqueduct
(*c*) level crossing (*d*) canal siphon.

Q. 292. The drainage water is sometimes allowed to join the canal water to augment canal supplies, through a hydraulic structure, called a :

(*a*) canal outlet (*b*) canal inlet
(*c*) module (*d*) level crossing.

Q. 293. The DBL of a canal and the HFL of a drain at their crossing point are, respectively, 216 m and 214 m. The adopted cross-drainage work here, will be a :

(*a*) super passage (*b*) siphon
(*c*) aqueduct (*d*) siphon aqueduct.

Q. 294. Pinpoint the correct statement in relation to the relative merits of the two cross-drainage works :

(*a*) a super passage is preferred to an aqueduct
(*b*) a canal-syphon is preferred to a super-passage
(*c*) a super passage is preferred to a canal syphon
(*d*) a siphon aqueduct is inferior to a canal siphon, and is seldom used.

Q. 295. A cross-drainage work is called a siphon, when it carries the canal water :
(*a*) below the drainage under pressure
(*b*) below the drainage at atmospheric pressure
(*c*) above the drainage at atmospheric pressure
(*d*) none of the above.

Q. 296. In a syphon aqueduct, the worst condition of uplift on the roof occurs, when:
(*a*) the canal and drainage are flowing full
(*b*) the canal is flowing full and there is no drainage discharge
(*c*) the canal is empty and drainage is flowing full
(*d*) none of the above.

Q. 297.* In an aqueduct provided with a pucca bottom floor, the uplift will occur :
(*a*) on the roof slab
(*b*) on the bottom floor
(*c*) on both the roof slab as well as the bottom floor
(*d*) no where, since the flow is free in the canal as well as in the drainage channel.

Q. 298.* In a syphon aqueduct, the worst condition of uplift on the floor occurs, when:
(*a*) the canal is full and the drainage empty, with watertable at drainage bed
(*b*) the canal is empty and the drainage full, with watertable at drainage bed
(*c*) the canal and drainage both full
(*d*) the canal is full and drainage empty, with watertable below the floor.

Q. 299. Point out the incorrect statement :
(*a*) aqueducts and super passages are usually not provided with pucca bottom floors
(*b*) aqueduct syphons and canal syphons are usually provided with pucca bottom floors
(*c*) in a level crossing, a cross regulator is provided on the canal below the crossing
(*d*) canals or drainage channels are usually flumed to affect economy at the sites of crossings, wherein the contraction transitions are not to be steeper than $22\frac{1}{2}^\circ$ and the expansion transitions not to be steeper than 30°.
(*e*) none of the above.

Q. 300. The head loss through a siphon barrel is usually given by Unwin's formula, by neglecting velocity of approach, as :

$H_L = \left(1 + f_1 + f_2 \cdot \frac{L}{R}\right)\frac{V^2}{2g}$, **where f_1 and f_2 respectively represent the coefficient of head losses due to :**

(*a*) barrel friction and entry (*b*) entry and barrel friction
(*c*) barrel friction and exit (*d*) exit and barrel friction.

Q. 301. A syphon aqueduct is constructed at a canal crossing site, where drainage HFL was 212.2 m, by allowing an afflux of 0.4 m at high flood discharge. At this site, the water level, downstream of the crossing at the same high flood, will be :
(*a*) 212.2 m
(*b*) 212.6 m
(*c*) 211.8 m
(*d*) none of them, as it cannot be predicted.

Q. 302. The following data is available at the proposed site of a canal crossing :

Item	Drain	Canal
B.L. (m)	252.2	248.0
FSL/HFL (m)	253.2	253.0
Discharge (*cumecs*)	2	400

The most appropriate and economical cross-drainage work at the above site will be :

(*a*) an aqueduct (*b*) a super passage
(*c*) a syphon aqueduct (*d*) a syphon.

CHAPTER 15

Q. 303. Culverts are all those road bridges, whose spans are up to :

(*a*) 6 m (*b*) 8 m
(*c*) 10 m (*d*) 12 m.

Q. 304.* A road bridge is proposed to be constructed across an alluvial stream. The design discharge at the bridge site is estimated to be 2500 m^3/s. It is decided to adopt a total effective waterway equal to 300 m for this bridge, as against the regime width of 237.5 m, due to afflux considerations. If the HFL at the site is at RL 214.0 m, then the RL of the open foundation can be suggested at :

(*a*) 206.6 m (*b*) 207.2 m
(*c*) 208.5 m (*d*) 210.2 m.

Q. 305. The relation between Scour depth (*R*) and Regime scour depth (R_e) for an alluvial river, having regime width (W) and contracted width (B), is given by :

(*a*) $R = R_e \left(\frac{W}{B}\right)^{0.61}$ (*b*) $R = R_e \left(\frac{B}{W}\right)^{0.61}$

(*c*) $R = R_e \left(\frac{W}{B}\right)^{1/3}$ (*d*) $R = R_e \left(\frac{B}{W}\right)^{1/3}$

Q. 306. The Lacey's scour depth R, given by equation $R = 0.473\left(\frac{Q}{f}\right)^{1/3}$, is applicable, when the river width equals :

(*a*) actual river width (*b*) regime width
(*c*) any contracted width (*d*) all of the above.

Q. 307. The Lacey's scour depth R, given by the equation $R = 1.35\left(\frac{q^2}{f}\right)^{1/3}$ is applicable, when the river width equals :

(*a*) regime width (*b*) actual river width
(*c*) any contracted width (*d*) all of the above.

Q. 308. Proposed RCC road bridge, having total height of its abutments or piers from top to bottom as H, can be proposed to have economical spans, each equal to :

(*a*) $\frac{2}{3}$H (*b*) H
(*c*) 1.2 H (*d*) 1.5 H.

Q. 309. A bridge culvert is to be constructed across a proposed lined canal. The afflux at this site for a contracted canal width, can be computed by :
(*a*) Unwin's formula (*b*) Broad-crested weir formula
(*c*) Orifice formula (*d*) Molesworth formula.

Q. 310. While going to Tirupati Tirumala Devasthanam from Madras, highway road is found to be lowered at several places, over which natural drainage flow during rainy season. Such dips in the roads, are called :
(*a*) freeways (*b*) road dips
(*c*) causeways (*d*) broadways.

CHAPTER 16

Q. 311. An irrigation project is classified as a major project, when the culturable command involved in the project, is more than :
(*a*) 2,000 hectares (*b*) 5,000 hectares
(*c*) 10,000 hectares (*d*) none of the above.

Q. 312. A minor irrigation scheme, involves command area, equal to or less than :
(*a*) 100 hectares (*b*) 500 hectares
(*c*) 1000 hectares (*d*) 2000 hectares.

Q. 313. Energy is required in the utilisation of :
(*a*) ground water (*b*) surface water
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 314. In an underground profile, the zone of aeration does not include :
(*a*) soil water (*b*) capillary water
(*c*) ground water (*d*) none of them.

Q. 315. Ground water is found to occur in the geological formation, which is :
(*a*) porous (*b*) permeable
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 316. The property of a geological formation, which represents its water storage capacity, is :
(*a*) permeability (*b*) porosity
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 317. The zone of aeration in a ground profile consists of :
(*a*) capillary fringe (*b*) soil water zone
(*c*) intermediate zone (*d*) all of them.

Q. 318. The zone of aeration in a ground profile does not include :
(*a*) capillary zone (*b*) soil water zone
(*c*) intermediate zone (*d*) saturation zone.

Q. 319. The area in our country, which is irrigated by ground water, as a fraction of the total irrigated area, is approximately equal to :
(*a*) $\frac{1}{2}$ (*b*) $\frac{2}{3}$
(*c*) $\frac{3}{4}$ (*d*) $\frac{1}{4}$.

Q. 320. The rate of flow of water through ground strata, can be estimated by :
(*a*) Manning's formula (*b*) Strickler's formula
(*c*) Dupuit's formula (*d*) Darcy's formula.

Q. 321. Darcy's law is valid, when the flow is :
(*c*) laminar (*b*) turbulent
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 322. The relation between Transmissibility(T) and Permeability(K) for an aquifer of depth d, is :
(*a*) $K = Td$ (*b*) $T = K \cdot d$
(*c*) $T = K \cdot \log d$ (*d*) $T = \ln (Kd)$.

Q. 323.* The geological formation, which yields only insignificant quantity of ground water, is an :
(*a*) aquifer (*b*) aquifuse
(*c*) aquiclude (*d*) aquitard.

Q. 324.* The zeological formation, which may contain water, but does not yield any, is an :
(*a*) aquifer (*b*) aquifuse
(*c*) aquiclude (*d*) aquitard.

Q. 325.* The zeological formation, which contains and readily yields water to our tubewells, is an :
(*a*) aquifuse (*b*) aquitard
(*c*) aquiclude (*d*) aquifer.

Q. 326. The capillary fringe water is also called :
(*a*) suspended water (*b*) vadose water
(*c*) gravity water (*d*) all of the above.

Q. 327. The quantum of water contained in the soil pores, which cannot be extracted by gravity drainage, is called :
(*a*) pellicular water (*b*) capillary water
(*c*) hygroscopic water (*d*) none of these.

Q. 328. The cum of ground water, which can be extracted by gravity drainage from a soil stratum, when expressed as percentage fraction of the cum of the soil stratum, is called :
(*a*) pellicular water (*b*) available water
(*c*) specific yield (*d*) field capacity.

Q. 329. Field capacity of a ground aquifer, equals :
(*a*) specific yield (*b*) 100—specific yield
(*c*) $\frac{100}{\text{specific yield}}$ (*d*) none of the above.

Q. 330. Specific retention of ground water is larger in :
(*a*) coarse grained soils (*b*) fine grained soils.

Q. 331. Water wells excavated through unconfined aquifers are known as :
(*a*) gravity wells (*b*) water table wells
(*c*) non-artesian wells (*d*) all of them.

Q. 332. Water wells excavated through confined aquifers are known as :
(*a*) artesian wells (*b*) pressure wells
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 333. In the case of a flowing well, the piezometric surface is always :
(*a*) below the ground level (*b*) above the ground level
(*c*) at the ground level (*d*) none of the above.

Q. 334. The line joining the static water levels in several wells, excavated through confined aquifer, is known as the :
(*a*) cone of depression (*b*) piezometric surface
(*c*) perched water-table (*d*) hypsometric curve.

Q. 335. If the piezometric surface along an unconfined aquifer declines to a level below the top of the aquifer, then the aquifer at this point is called :
(*a*) a perched aquifer (*b*) a leaky aquifer
(*c*) a flowing aquifer (*d*) an unconfined aquifer.

Q. 336.* The permeability of an aquifer (m/day) will :
(*a*) increase with increase in temperature of water flowing through the aquifer
(*b*) decrease with increase in temperature of water flowing through the aquifer
(*c*) not get affected by the change in temperature of water flowing through the aquifer
(*d*) increase upto 20°C and then decrease with the increase in the temperature of water flowing through the aquifer.

Q. 337.* The component of the permeability of a porous medium, which is usually expressed in units of darcy or m^2, is called :
(*a*) specific permeability (*b*) intrinsic permeability
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 338. The discharge per unit drawdown at a well is known as :
(*a*) specific yield (*b*) specific retention
(*c*) specific capacity (*d*) specific storage.

Q. 339. Storage coefficient (A) divided by aquifer depth (*d*) is known as :
(*a*) specific storage (*b*) specific capacity per unit depth of aquifer
(*c*) specific yield. (*d*) specific retention.

Q. 340.* Coefficient of storage (A) has the dimensions of :
(*a*) L^3 (*b*) L^2
(*c*) L (*d*) dimensionless.

Q. 341. The units of specific capacity of a well are :
(*a*) m^3/sec (*b*) m^2/sec
(*c*) m/sec (*d*) no units.

Q. 342. Specific capacity of a well is :
(*a*) constant over time after commissioning of the well
(*b*) increases with time after start of pumping
(*c*) decreases with time after start of pumping
(*d*) may increase or decrease, depending upon a particular aquifer.

Q. 343. Standard specific capacity of a well is the well discharge per unit of drawdown. This value of discharge should be obtained at :
(*a*) any drawdown
(*b*) a particular drawdown of 3 m
(*c*) the first 1 m of drawdown
(*d*) none of the above.

Q. 344.* The performance of a well is measured by its :
(*a*) specific capacity (*b*) specific yield
(*c*) storage coefficient (*d*) all of the above.

Q. 345. The clogging of well screens and consequent reduction in pump efficiency is indicated by :
(*a*) high value of well loss (*b*) low value of well loss
(*c*) variable value of well loss (*d*) none of the above.

Q. 346. The water level in a confined well :
(*a*) increases with increase in atmospheric pressure
(*b*) decreases with increase in atmospheric pressure
(*c*) does not undergo any change with change in the atmospheric pressure.
(*d*) all of the above are possible.

Q. 347. The water level in an unconfined well :
(*a*) increases with increase in atmospheric pressure
(*b*) decreases with increase in atmospheric pressure
(*c*) does not undergo any change with change in the atmospheric pressure
(*d*) all of the above are possible.

Q. 348.* "Specific yield" for an unconfined aquifer is :
(*a*) greater than porosity (*b*) less than porosity
(*c*) equal to porosity (*d*) unrelated to porosity. *(GATE 1989)*

Q. 349. A perched aquifer is essentially found within :
(*a*) an unconfined aquifer (*b*) a confined aquifer
(*c*) an aquiclude (*d*) none of these.

Q. 350. The most widely used type of a deep state tubewell in India is:
(*a*) cavity well (*b*) strainer tubewell
(*c*) slotted pipe gravel packed tubewell
(*d*) none of them.

Q. 351. The general average yield from standard tubewells is of the order of :
(*a*) 5 L/s (*b*) 50 L/s
(*c*) 500 L/s (*d*) 5000 L/s.

Q. 352. The types of pumps used in tubewells are :
(*a*) submergible pumps (*b*) centrifugal pumps
(*c*) turbine pumps (*d*) all of these.

Q. 353. The appropriate life of a tubewell in India is of the order of :
(*a* 1 year (*b*) 2 years
(*c*) 5 years (*d*) 15 years.

Q. 354. Corrosion of tubewell pipes may cause :
(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly corrosive acidic water
(*d*) none of the above.

Q. 355. Incrustation of the tubewell pipes may lead to :
(*a*) reduced discharge from the tubewell
(*b*) excessive discharge of sand with water
(*c*) discharge of highly alkaline water
(*d*) none of the above.

Q. 356. The efficiency of the centrifugal pumps, commonly used for lifting water from wells, may be of the order of :
(*a*) 30% (*b*) 65%
(*c*) 85% (*d*) 95%.

Q. 357. Deep tubewells in soft alluviums can be best drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 358. Deep tubewells in hard boulder-alluviums can be best drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 359. Bored tubewells in rocky consolidated formations, as are encountered in South Indian States of our country, are usually drilled by :
(*a*) rotary drilling rigs
(*b*) percussion drilling rigs
(*c*) down the hole hammer (DTH) rigs
(*d*) none of the above.

Q. 360. Shallow tubewells upto 70 m depth in alluvial soils, are usually drilled by :
(*a*) light rigs (*b*) water jet methods
(*c*) cable tool drills (*d*) hand boring devices
(*e*) all of the above.

CHAPTER 17

Q. 361. Aswan dam on Nile river in Egypt was perhaps the first modern dam of the world. Its construction was completed in the year :
(*a*) 1664 (*b*) 1798
(*c*) 1902 (*d*) 1931.

Q. 362. The only modern dam among the following, which has failed, is :
(*a*) Koyna dam in India
(*b*) Vega de Tera dam in spain
(*c*) Mauvoisin dam in Switzerland
(*d*) Trinity dam in California, USA.

Q. 363. Earthen dams are :
(*a*) rigid dams (*b*) non rigid dams
(*c*) overflow dams (*d*) diversion dams.

Q. 364. When sand and gravel foundation strata is available at a proposed dam site of moderate height, the dam may be of the type :
(*a*) earthen dam or rockfill dam
(*b*) masonry gravity dam
(*c*) double arch dam
(*d*) concrete gravity dam.

Q. 365. The prominent modern dam, which has changed its name several times, is :
(*a*) Nagarjuna sagar dam (*b*) Hoover dam
(*c*) Bhakra dam (*d*) Idduki dam.

Q. 366. The highest concrete gravity dam of the world is Grand Dixence dam is Switzerland, whose height is :
(*a*) 325 m (*b*) 317 m
(*c*) 285 m (*d*) 272 m.

Q. 367. The only arch dam of our country is Idduki dam across Periyar river in Kerala State, whose height is :
(*a*) 81 m (*b*) 143 m
(*c*) 149 m (*d*) 169 m.

Q. 368. The highest concrete gravity dam of India, which is also the 2nd highest such dam of the world, is :
(*a*) 285 m in height (*b*) 256 m in height
(*c*) 226 m in height (*d*) 206 m in height.

Q. 369. Bhakra dam of our country is located in the State of :
(*a*) Punjab (*b*) Himachal Pradesh
(*c*) Uttar Pradesh (*d*) Madhya Pradesh.

Q. 370.* The highest stone masonry gravity dam of India as well as of the world, is:
(*a*) Ramganga dam (*b*) Nagarjuna Sagar dam
(*c*) Hirakud dam (*d*) Mettur dam.

CHAPTER 18

Q. 371. A dam reservoir, not provided with gate controls on its spillway and other sluices, is called a :
(*a*) detention basin (*b*) storage reservoir
(*c*) retarding basin (*d*) all of these.

Q. 372. A dam reservoir, catering to flood control, irrigation, and water supply, although basically designed for irrigation alone, is a :
(*a*) multipurpose reservoir (*b*) single purpose reservoir
(*c*) distribution reservoir (*d*) none of the above.

Q. 373. A hydel power project has been envisaged to serve the water supply and irrigation needs of the area at its inception stage. The dam reservoir, so constructed, will be known as :
(*a*) multipurpose reservoir (*b*) single purpose reservoir
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 374. The 'surcharge storage' in a dam reservoir is the volume of water stored between :
(*a*) minimum and maximum reservoir levels
(*b*) minimum and normal reservoir levels
(*c*) normal and maximum reservoir levels
(*d*) none of the above.

Q. 375. The 'useful storage' in a dam reservoir is the volume of water stored between:
(*a*) minimum and maximum reservoir levels
(*b*) minimum and normal reservoir levels
(*c*) normal and maximum reservoir levels
(*d*) none of the above.

Q. 376. The 'dead storage' in a dam reservoir, is the available volume for collection of silt and sediment, between :
(*a*) bed level of the reservoir and minimum reservoir level
(*b*) bed level of the reservoir and the silt level in the reservoir
(*c*) bed level of the reservoir and the normal pool level
(*d*) none of the above.

Q. 377. 'Bank storage' in a dam reservoir :
(*a*) increases the computed reservoir capacity
(*b*) decreases the computed reservoir capacity
(*c*) sometimes increases and sometimes decreases the computed reservoir capacity
(*d*) has no effect on computed reservoir capacity.

Q. 378. The water stored in a reservoir below the minimum pool level is called :
(*a*) valley storage (*b*) bank storage
(*c*) surcharge storage (*d*) dead storage.

Q. 379. For a flood control reservoir, the effective storage is equal to :
(*a*) useful storage + valley storage
(*b*) useful storage + surcharge storage – valley storage
(*c*) useful storage + surcharge storage + valley storage
(*d*) useful storage – valley storage.

Q. 380. The volume V of water stored in a reservoir in a depth y above a given datum, can be easily represented by an equation of the type :
(*a*) $V = [\alpha + \beta y + \gamma y^2]$ where α, β, γ are constants
(*b*) $V = [\alpha y + \beta y^2 + \gamma y^3] + K$ K is reservoir capacity up to given datum
(*c*) neither (*a*) nor (*b*).
(*d*) both (*a*) and (*b*).

Q. 381. Average yield of a storage reservoir is the arithmetic average of its :
(*a*) firm yields over a long period
(*b*) secondary yields over a long period
(*c*) firm and secondary yields over a long period
(*d*) none of the above.

Q. 382. If A_1 and A_2 are surveyed and known areas at a reservoir site at elevations of E_1 and E_2, respectively ; then the area, A, at any intermediate elevation E, is interpolated as :

(*a*) $\sqrt{A} = \sqrt{A_1} + \dfrac{E - E_1}{E_2 - E_1}\left(\sqrt{A_2} - \sqrt{A_1}\right)$

(*b*) $\sqrt{A} = \sqrt{A_1} + \dfrac{E + E_1}{E_2 + E_1}\left(\sqrt{A_2} + \sqrt{A_1}\right)$

(*c*) $A = \sqrt{A_1} + \dfrac{E - E_1}{E_2 - E_1}\left[\sqrt{A_2} - \sqrt{A_1}\right]^2$

(*d*) none of the above.

Q. 383. While planning a water supply reservoir, as compared to an irrigation reservoir, the design yield may be kept :
(*a*) higher (*b*) lower
(*c*) equal
(*d*) lower or higher, as per designer's discretion.

Q. 384. Yield of a reservoir represents :
(*a*) the inflow into the reservoir
(*b*) the capacity of the reservoir
(*c*) the outflow demand on the reservoir
(*d*) none of the above.

Q. 385. The capacity of a storage reservoir can be decided by using :
(*a*) the mass curve of inflow
(*b*) the mass curve of outflow
(*c*) by both (*a*) and (*b*) together
(*d*) none of the above.

Q. 386. The maximum quantity of water that is estimated to remain available in a storage reservoir for supply, even during worst dry periods, is known as its:
(*a*) firm yield (*d*) design yield
(*c*) reservoir yield (*d*) primary yield.

Q. 387. Design yield of a storage reservoir is kept :
(*a*) higher than its firm or safe yield
(*b*) lower than its firm or safe yield
(*c*) equal to its firm or safe yield
(*d*) higher or lower than the firm yield, depending upon the designer's intuition.

Q. 388. The amount of water stored in a river channel without any artificial storage, is known as :
(*a*) bank storage (*b*) river storage
(*c*) valley storage (*d*) dead storage.

Q. 389. With the reduction in reservoir capacity over the passage of time, the trap efficiency
(*a*) increases
(*b*) decreases
(*c*) remains unaffected
(*d*) may increase or decrease, depending upon the reservoir characteristics.

Q. 390. Trap efficiency of a storage reservoir is defined as :
(*a*) $\frac{\text{Total annual sediment inflow}}{\text{Reservoir capacity}}$
(*b*) $\frac{\text{Total sediment deposited in a given period}}{\text{Total sediment inflow in that period}}$
(*c*) $\frac{\text{Total annual sediment deposited in the reservoir}}{\text{Dead storage capacity of the reservoir}}$
(*d*) none of the above.

Q. 391. Capacity-inflow ratio for a storage reservoir is defined as :
(*a*) $\frac{\text{reservoir capacity}}{\text{average annual flood inflow}}$
(*b*) $\frac{\text{reservoir capacity}}{\text{average annual sediment inflow}}$
(*c*) $\frac{\text{dead storage capacity of the reservoir}}{\text{average annual sediment deposited}}$
(*d*) none of the above.

Q. 392. With the increase in its capacity-inflow ratio, the trap efficiency of a reservoir:
(*a*) increases
(*b*) decreases
(*c*) remains unchanged
(*d*) may increase or decrease depending on the reservoir characteristics.

Q. 393. The capacity-inflow ratio for a reservoir :
(*a*) is a constant factor overtime
(*b*) increases with time
(*c*) decreases with time
(*d*) may increase or decrease with time.

Q. 394. As the height of a proposed dam is increased, the cost per unit of storage :
(*a*) increases
(*b*) decreases
(*c*) initially increases and then decreases
(*d*) initially decreases and then increases.

Q. 395. "Economical height of a Dam" is that height, for which the :
(*a*) cost per unit of storage is minimum
(*b*) benefit cost ratio is maximum
(*c*) net benefits are maximum
(*d*) none of the above.

Q. 396.* If 20% of the reservoir capacity is earmarked for dead storage in a storage reservoir of 30 M cum; and the average annual silt deposition in the reservoir is 0.1 M cum, then the useful life of the reservoir will start reducing after :
(*a*) 60 years (*b*) 120 years
(*c*) 240 years (*d*) 300 years.

Q. 397. The appropriate value of the estimated average life of a dam reservoir is :
(*a*) 25 years (*b*) 50 years
(*c*) 75 years (*d*) none of these.

CHAPTER 19

Q. 398. The vertical component of the earthquake wave, which produces adverse effects on the stability of a dam, is, when it is acting in;
(*a*) upward direction (*b*) downward direction
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 399. The horizontal component of an earthquake wave, producing instability in a dam is the one, which acts :
(*a*) towards the reservoir (*b*) towards the dam
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 400. The vertical downward earthquake acceleration, $\alpha_v = 0.1$ g, acting on a gravity dam, will :
(*a*) increase the resisting weight of the dam by 10%
(*b*) decrease the resisting weight of the dam by 10%
(*c*) increase the uplift by 10%
(*d*) none of the above.

Q. 401. A gravity dam is subjected to hydro dynamic pressure, caused by :
(*a*) the rising waters of the reservoir when a flood wave enters into it
(*b*) the rising waves in the reservoir due to high winds
(*c*) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards the reservoir.
(*d*) the increase in water pressure, momentarily caused by the horizontal earthquake, acting towards the dam.

Q. 402. In a concrete gravity dam with a vertical upstream face, the stabilising force is provided by the :
(*a*) weight of the dam
(*b*) the water supported against the upstream slope
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 403. In a concrete gravity dam, with a sloping upstream face, the resisting force is provided by the :

(*a*) weight of the dam
(*b*) weight of the water supported on the upstream slope
(*c*) both (*a*) and (*b*)
(*d*) none of these.

Q. 404. The factor among the following, which does not try to destabilise a masonry gravity dam, is :

(*a*) water seeping below the foundation of the dam
(*b*) generation of waves by high winds
(*c*) deposition of silt in dead storage zone of the reservoir
(*d*) water standing against the downstream face of the dam
(*e*) water standing against the upstream face of the dam.

Q. 405*. The horizontal destabilising force caused by the formation of waves in a storage reservoir having a fetch of 40 km, due to high wind of 160 km/h, is about :

(*a*) 30 kN (*b*) 60 kN
(*e*) 90 kN (*d*) 130 kN.

Q. 406*. According to IS 6512—1984, the wind set up in a reservoir having maximum fetch of F km, and average reservoir depth along this fetch length as D m, as caused by the wind velocity of V km/h, is given by :

(*a*) $\frac{V^2 F}{62000 D}$ (*b*) $\frac{V^2 F}{D}$

(*c*) $\frac{V^2 F}{6200 D}$ (*d*) $\frac{V^2 F}{620 D}$.

Q. 407*. A reservoir, extending 20 km upstream and having a design wind velocity of 100 km/h, should be provided with a free-board of :

(*a*) 1.2 m (*b*) 1.6 m
(*c*) 2.5 m (*d*) 3 m.

Q. 408. Development of tensile stresses in a concrete or masonry gravity dam are usually not allowed, because it may lead to development of tension cracks, ultimately causing failure of the structure, by :

(*a*) excessive seepage
(*b*) excessive tensile stresses
(*c*) excessive compressive stresses
(*d*) none of the above.

Q. 409. The bottom portion of a concrete or a masonry gravity dam is usually stepped, in order to :

(*a*) increase the overturning resistance of the dam
(*b*) increase the shear strength at the base of the dam
(*c*) decrease the shear stress at the base of the dam
(*d*) none of the above.

Q. 410. The governing compressive stress in a concrete gravity dam, which should not be allowed to exceed the permissible value of about 3000 kN/m^2, while analysing full reservoir case, is :

(*a*) the vertical maximum stress at the toe
(*b*) the major principal stress at the toe
(*c*) the shear stress at the toe
(*d*) none of the above.

Q. 411. The two dimensional stability analysis of gravity dams proves better for U-shaped valleys than for *V*-shaped valleys, because the transverse joints in the dam body are generally :

(*a*) not grouted in *U*-shaped valleys, but are keyed together in *V*-shaped valleys

(*b*) not grouted in *V*-shaped valleys, but are keyed together in *U*-shaped valleys.

Q. 412. The small openings made in the huge body of a concrete gravity dam, such as sluices and inspection galleries, can be assumed to be causing only local effects without any appreciable effect on the distribution of stresses, as per the principle of :

(*a*) Laplace (*b*) St. Venant

(*c*) Reynold (*d*) St. Francis.

Q. 413. For usuall values of permissible compressive stress and specific gravity of concrete, a high concrete gravity dam is the one, whose height exceeds :

(*a*) 48 m (*b*) 70 m

(*c*) 88 m (*d*) none of these.

Q. 414*. An elementary triangular concrete gravity dam, supporting 60 m height of reservoir water and full uplift, should have a minimum base width equal to:

(*a*) 36 m (*b*) 39 m

(*c*) 51 m (*d*) 61 m.

Q. 415. Inorder to economise on the provided section of a concrete gravity dam, attempts are made to reduce the uplift, by :

(*a*) providing drainage gallery to collect seepage water

(*b*) constructing cut-off under upstream face

(*c*) pressure grouting in dam foundation

(*d*) constructing drainage channels between the dam and its foundation

(*e*) all of the above methods.

Q. 416. Transverse joints in concrete gravity dams are the :

(*a*) horizontal construction joints at each lift height

(*b*) vertical construction joints of full height and width

(*c*) diagonal construction joints for torsion

(*d*) none of the above.

Q. 417. Leakage through the transverse joints in a gravity dam is prevented by :

(*a*) shear keys (*b*) key ways

(*c*) water stops (*d*) none of these.

Q. 418. Shear key is several times provided between the bottom of a masonry or concrete gravity dam and its foundation, to increase the frictional resistance of the dam against sliding. This key is usually provided :

(*a*) near the toe (*b*) near the heel

(*c*) near the individual seams in the bed rock

(*d*) none of the above.

CHAPTER 20

Q. 419. Point out the correct statement with reference to earthen dams :

(*a*) these dams are very costly as compared to other types

(*b*) they are less susceptible to failure as compared to rigid dams

(*c*) they can be constructed almost on every type of foundation

(*d*) highly skilled labour is generally not required.

Q. 420. The most preferred type of an earthen dam section is the one, in which the:
(*a*) entire embankment is made of one type of soil
(*b*) inner embankment is made of highly porous soil, surrounded by the outer shell of highly impervious soil, both separated by transition filter material of mediocre permeability
(*c*) inner embankment is made of highly impervious soil surrounded by the outer shell of highly pervious soil, both separated by transition filter material of mediocre permeability
(*d*) none of the above.

Q. 421. When seepage takes place through the body of an earthen dam, it leads to :
(*a*) development of pore pressures in the dam body
(*b*) reduction in the shear strength of the dam
(*c*) reduction in the developed shear stresses in the dam
(*d*) (*a*) and (*b*) both
(*e*) (*a*), (*b*) and (*c*) all.

Q. 422. The most preferred soil for the central impervious core of a zoned embankment type of an earthen dam, is
(*a*) highly impervious clay (*b*) highly pervious gravel
(*c*) coarse sand (*d*) clay mixed with fine sand.

Q. 423. Pure clayey soils are generally not preferred for the central impervious cores of zoned type of earthen dams, because :
(*a*) clays are highly impervious
(*b*) clays are highly pervious
(*c*) clays are susceptible to cracking
(*d*) none of the above.

Q. 424. During the construction of an earthen dam by hydraulic fill method, development of pore pressures become important in the :
(*a*) central impervious core (*b*) pervious outer shell
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 425. The process of laying and compacting earth in layers by power rollers under Optimum Moisture Conditions (OMC) for construction of earthen dams, is known as :
(*a*) rolled fill method (*b*) hydraulic fill method
(*c*) OMC method (*d*) none of the above.

Q. 426. Coordination between field and design engineers to ensure continuous field observations and modifications in design during construction, is more importantly required in case of :
(*a*) concrete gravity dams (*b*) masonry gravity dams
(*c*) arch dams (*d*) earthen dams.

Q. 427. When the water level standing against an earthen embankment, suddenly falls down, then there is an iminent risk of sliding failure, to the :
(*a*) upstream slope (*b*) downstream slope
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 428. When the reservoir is full, the slope which is most likely to slide, is :
(*a*) the upstream slope (*b*) the downstream slope
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 429. Provision of horizontal berms at suitable vertical intervals may be provided in the downstream face of an earthen dam, in order to :
(*a*) allow the movement of cattle
(*b*) allow the inspection vehicles to move
(*c*) reduce the erosion caused by the flowing rain water
(*d*) none of the above.

Q. 430. The only provision among the following, which can help control the seepage through the body of an earthen dam and, thus, to keep the phreatic line well within the dam width, is :
(*a*) upstream impervious cutoff
(*b*) drain trench along the downstream toe
(*c*) relief wells
(*d*) chimney drain.

Q. 431*. During the maintenance of an earthen dam, the apparent seepage through the foundation of the dam is best taken care of, by providing :
(*a*) a chimney drain
(*b*) a rock toe
(*c*) a drain trench along the downstream toe
(*d*) an upstream impervious cutoff.

Q. 432. The base width of a rock fill dam, incomparison to that of an earthen dam, is :
(*a*) much larger
(*b*) much smaller
(*c*) sometimes larger sometimes smaller
(*d*) almost equal.

Q. 433. On moderate foundations, and particularly in seismic areas, the type of dam which can preferably be considered for construction, is :
(*a*) masonry gravity dam (*b*) earthen dam
(*c*) rock fill dam (*d*) arch dam.

CHAPTER 21

Q. 434. The 'safety valve' of a dam is its :
(*a*) drainage gallery (*b*) inspection gallery
(*c*) spillway (*d*) outlet sluices.

Q. 435. A ungated spillway starts functioning, as soon as the water level in the reservoir, crosses the :
(*a*) maximum reservoir level
(*b*) minimum reservoir level
(*c*) maximum conservation level
(*d*) none of the above.

Q. 436. The top line of an ogee spillway, which is kept at RL equal to the full reservoir level, is precisely called :
(*a*) the crest (*b*) the apex of the crest
(*c*) the corbel (*d*) the chute.

Q. 437. When the gated sluices are provided through the body of a dam spillway, then the ogee spillway structure shall normally be of :
(*a*) corbel type (*b*) non corbel type
(*c*) either (*a*) or (*b*) (*d*) none of these.

Q. 438. The effective length of an ogee spillway will be different from its actual length, in case of :
(*a*) gated spillway
(*b*) ungated spillway
(*c*) both gated as well as ungated spillway
(*d*) none of these.

Q. 439. Which one of the following spillways is least suited to earthen dams ?
(*a*) ogee spillway (*b*) side channel spillway
(*c*) chute spillway (*d*) shaft spillway.

Q. 440. The spillway, which can be adopted with ease on gravity as well as earthen dams, is :
(*a*) ogee spillway
(*b*) chute spillway
(*c*) both ogee as well as chute spillway
(*d*) none of these.

Q. 441. The famous Bhakra dam of our country has been provided with :
(*a*) trough spillway (*b*) ogee spillway
(*c*) shaft spillway (*d*) syphon spillway.

Q. 442. Hydraulic jump is widely used for dissipation of energy in :
(*a*) ogee spillways (*b*) trough spillways
(*c*) side channel spillways (*d*) all of these.

Q. 443. The surplus reservoir water, after spilling over the spillway crest, flows on the chute :
(*a*) parallel to the crest in a trough spillway
(*b*) parallel to the crest in a side channel spillway
(*c*) perpendicular to the crest in a side channel spillway
(*d*) obliquely to the crest in a chute spillway.

Q. 444. When the crest of an ogee spillway is designed to be in accordance with the lower nappe of a free falling water jet over a duly ventilated sharp crested weir, then theoretically :
(*a*) the pressure on the spillway crest will always be zero (*i.e.* atmospheric pressure)
(*b*) the pressure on the spillway crest will be zero at design head only
(*c*) the pressure on the spillway crest will always be negative
(*d*) the pressure on the spillway crest will always be positive.

Q. 445. If the operating head on an ogee spillway is more than the design head, then:
(*a*) the pressure on the crest will be zero
(*b*) the pressure on the crest will be negative, causing cavitation
(*c*) the pressure on the crest will be positive
(*d*) the discharge coefficient of the spillway will be reduced.

Q. 446. In computing the spilling capacity of high ogee spillways, the velocity head is usually :
(*a*) very small, and hence neglected
(*b*) very large, and hence cannot be neglected
(*c*) neither (*a*) nor (*b*).

Q. 447. In the functioning of an ogee spillway, the operating head :
(*a*) frequently exceeds the design head
(*b*) rarely exceeds the design head
(*c*) never exceeds the design head
(*d*) has no connection with the design head.

Q. 448. The discharge passing over an ogee spillway, per unit length of its appex line, is proportional to :
(*a*) H (*b*) H^2
(*c*) $H^{1/2}$ (*d*) $H^{3/2}$.

where H is the head over the appex of its crest.

Q. 449*. If a flood enters a dam reservoir at F.R.L., the efficiency of its ogee spillway will :
(*a*) increase with the increasing head
(*b*) reduce with the increasing head
(*c*) remain constant with the increasing head
(*d*) vary with the increasing head.

Q. 450. The W.E.S. downstream profile of an ogee spillway can be represented by an equation :

$$\left(\frac{x}{H_d}\right)^n = K\left(\frac{y}{H_d}\right);$$

where the coefficients K and n for a vertical upstream spillway would, respectively, be :
(*a*) 2.0 and 1.85 (*b*) 0.5 and 1.85
(*c*) 2.0 and 0.85 (*d*) 0.5 and 0.85.

where (x, y) are the coordinates on the crest profile with crest appex as origin.

Q. 451. The portion of a chute spillway, which is known as its control structure, is :
(*a*) low ogee weir
(*b*) chute channel
(*c*) approach channel leading the water from the reservoir to the ogee weir.
(*d*) stilling basin at its bottom.

Q. 452. The spillway, which can be called as an 'overflow spillway', is essentially :
(*a*) an ogee spillway (*b*) shaft spillway
(*c*) chute spillway (*d*) syphon spillway.

Q. 453. The spillway, which may sometimes be called a 'waste weir', is :
(*a*) an ogee spillway (*b*) a trough spillway
(*c*) a shaft spillway (*d*) all of the above.

Q. 454. The spillways involving weir type spills, giving increased discharge with the increase in reservoir level, are :
(*a*) ogee spillways (*b*) chute spillways
(*c*) side channel spillways (*d*) all of these.

Q. 455. A shaft spillway is located :
(*a*) inside the body of a gravity dam
(*b*) inside the upstream reservoir
(*c*) inside the downstream reservoir
(*d*) on side flanks of the main dam.

Q. 456. The syphons installed within a gravity dam, to spill the surplus reservoir water, are known as :

(*a*) hooded type syphon spillway
(*b*) tilted outlet type syphon spillway
(*c*) both (*a*) and (*b*)
(*d*) none of these.

Q. 457. The syphons installed over overflow dams, constitute what are known as :

(*a*) tilted outlet type of syphon spillways
(*b*) hooded type of syphon spillways
(*c*) discharge carriers
(*d*) baby syphons.

Q. 458. Syphons installed over an overflow spillway will :

(*a*) decrease the effectiveness of the spillway
(*b*) increase the effectiveness of the spillway
(*c*) not affect the effectiveness of the spillway
(*d*) all the above are possible.

Q. 459. The only spillway among the following, through which the discharge does not increase as fast as it increases in all others, is :

(*a*) chute spillway (*b*) side channel spillway
(*c*) ogee spillway (*d*) shaft spillway.

Q. 460. The only spillway among the following, through which the discharge is almost at its capacity rate, even from the start of its functioning, is :

(*a*) chute spillway (*b*) side channel spillway
(*c*) ogee spillway (*d*) syphon spillway.

Q. 461*. An air vent is provided at F.R.L. to break the syphoning action at that level, in a :

(*a*) hooded type of syphon spillway
(*b*) tilted outlet type of syphon spillway
(*c*) saddle syphon spillway
(*d*) all of the above.

Q. 462. The energy dissipation at the toe of a spillway is affected basically by the use of hydraulic jump, in :

(*a*) a roller bucket
(*b*) a ski-jump bucket
(*c*) a sloping apron taken below the downstream river bed
(*d*) none of the above.

Q. 463. The most ideal condition for energy dissipation in the design of spillways is the one, when :

(*a*) the tail water rating curve lies above the jump rating curve at all discharges
(*b*) the tail water rating curve coincides with the jump rating curve at all discharges
(*c*) the tail water rating curve lies below the jump rating curve at all discharges
(*d*) the tail water rating curve either lies above or below the jump rating curve, depending upon the discharge.

Q. 464. When the tail water depths in the river, downstream of a spillway are quite low, such that the tail water curve, at all discharges, lies below the post jump depth curve, then the energy dissipation can be affected best by :

(*a*) a roller bucket (*b*) a ski-jump bucket
(*c*) either (*a*) or (*b*) (*d*) none of them.

Q. 465. Energy dissipation arrangements made at the bottom of an overfall spillway, usually depend upon the relative positions of tail water curve (T.W.C.) and jump height curve (J.H.C.) and also on Incoming Froude number. Such positions and suggested arrangements are given below in a haphazard manner, and you have to match them properly :

(*A*) T.W.C. coinciding with J.H.C. at all discharges	(*i*) ski-jump bucket at higher incoming Froude number (> 9); and sloping apron below the d/s river bed at lower froude numbers (4.5 to 9)
(*B*) T.W.C. lying below the J.H.C. at all discharges	(*ii*) a single horizontal apron at toe of the spillway at d/s river bed
(*C*) T.W.C. lying above the J.H.C. at all discharges	(*iii*) a roller bucket at higher Froude numbers, and sloping apron above the d/s river bed at lower values of *F*
(*D*) T.W.C. lying above the J.H.C. at low discharges, and below the J.H.C. at high discharges, and *vice versa.*	(*iv*) a sloping apron partly below and partly above the d/s river bed.

Q. 466. A troublesome and oscillating hydraulic jump is usually formed in flows, involving the incoming Froude number in the range of 2.5-4.5, which normally is met in cases of :

(*a*) weirs and barrages (*b*) overflow spillways of dams
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 467. A very steady and stable hydraulic jump is usually formed in flows, involving the approaching Froude number in the range of 4.5-9.0, which normally is met in cases of :

(*a*) weirs and barrages (*b*) overflow spillways of dams
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 468. In case of very high overflow spillways, the approaching Froude number of the flow at the toe of the spillway may exceed 9.0, bringing the hydraulic jump to be in the category of a 'strong jump'. In such a case, the energy dissipation arrangement preferably made at the toe of the spillway, may consists of :

(*a*) bucket type dissipator
(*b*) USBR stilling basin
(*c*) Indian standard stilling basin
(*d*) none of the above.

Q. 469. Standard USBR stilling basin II is useful for energy dissipation at the bottom of an overflow structure, if the approaching Froude number is :

(*a*) less than 4.5
(*b*) more than 4.5.

Q. 470. Standard stilling basin provided at the toe of a dam spillway for energy dissipation, is usually provided with auxiliary devices, like chute blocks and dentated sills, for the basic purpose of reducing the length of the stilling basin from about :

(*a*) $6y_2$ to $4y_2$ (*b*) $4y_2$ to $2y_2$

(*c*) $6y_2$ to $2y_2$ (*d*) $3y_2$ to y_2.

where y_2 is the post jump depth

Q. 471. The device which does not help in energy dissipation at the bottom of a hydraulic structure, over which water spills, is :

(*a*) chute block (*b*) dentated sill

(*c*) morning glory (*d*) none of them.

Q. 472. In a fixed roller of a spillway gate, the rollers are attached to the :

(*a*) gate (*b*) groove guide

(*c*) either (*a*) or (*b*) (*d*) none of them.

Q. 473. In vertical stoney spillway gate, the rollers are placed between the :

(*a*) gate and the u/s groove guide

(*b*) gate and the d/s groove guide

(*c*) either (*a*) or (*b*)

(*d*) none of them.

Q. 474. The spillway gate, which when lowered, can not be seen from a distance, is of the type :

(*a*) sliding gate (*b*) roller gate

(*c*) tainter gate (*d*) USBR drum gate.

Q. 475*. An ogee spillway of a concrete gravity dam having FRL of 325.0 m and MWL of 340.0 m, is provided with vertical gates between piers erected on the spillway. The effective length of the spillway is 60 m. The discharge through this spillway when gates are opened upto actual reservoir level of 331.0 m, will be :

(*a*) 850 cumecs (*b*) 1700 cumecs

(*c*) 2800 cumecs (*d*) none of them.

CHAPTER 22

Q. 476*. A 2 m diameter sluiceway at RL 300.0 m, is provided through a concrete overflow dam section to release water to the downstream, where tail water level is not more than 275.0 m. The discharge through this outlet, when the water level in the reservoir is at FRL of 330.0 m, is of the order of :

(*a*) 20 cumecs (*b*) 40 cumecs

(*c*) 50 cumecs (*d*) 80 cumecs.

Q. 477*. A 4 m diameter tunnel has been constructed through an earthen dam, with a bell mouth entry. The water levels on U/s and D/s sides of the dam are at RL 226 m and 210 m, respectively. The discharge through this outlet will be of the order of :

(*a*) 64 cumecs (*b*) 104 cumecs

(*c*) 170 cumecs (*d*) 216 cumecs.

Q. 478. A trash rack is not required at the entrance of a :

(*a*) syphon spillway (*b*) drum gate installation

(*c*) morning glory spillway (*d*) high head gate installation.

Q. 479. The outlet provided in a dam body, to release water for downstream water demand, is known as :

(*a*) spillway (*b*) sluiceway
(*c*) under-sluice (*d*) waterway.

Q. 480. Bar screens, used to cover dam outlets to prevent entry of debris or ice into the sluiceway conduits, are called :

(*a*) gate controlled ports (*b*) projecting collars
(*c*) trash racks (*d*) none of these.

Q. 481. Which of the following statement is false ?

(*a*) water can be withdrawn from any selected level of reservoir using dry intake towers
(*b*) Intake towers have to be designed for worst combination of hydrostatic, wind, earthquake, and wave forces
(*c*) Dry intake towers are lighter in construction as compared to wet intake towers
(*d*) none of the above.

Q. 482. A sluiceway conduit passing through an earthen dam has a length equal to L; and it has x projecting collars-each having projected length equal to x. The increased length of seepage path in this case is given by :

(*a*) $L + 2nx$ (*b*) $L + nx$
(*c*) $2nx$ (*d*) none of them.

Q. 483. Projecting collars are provided on sides of a rectangular tunnel of length (L), made through an earthen dam, to increase the seepage path. Their projection length (X) and numbers (N) are decided, as to provide increased seepage path equal to 2NX, where 2NX should generally be :

(*a*) greater than $\frac{L}{4}$ (*b*) greater than $\frac{L}{2}$
(*c*) greater than L (*d*) none of the above.

CHAPTER 23

Q. 484. The flow through Penstocks and Pressure conduits is, generally :

(*a*) laminar (*b*) turbulent
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 485. The head lost in the flow of water through a penstock pipe of given length:

(*a*) increases with the increase in flow velocity
(*b*) decreases with the increase in flow velocity
(*c*) decreases with the increase in the roughness of the pipe surface
(*d*) decreases with time.

Q. 486. Hoop's reinforcement is provided in cement concrete pressure pipes, in order to counteract the;

(*a*) water hammer pressure
(*b*) internal water pressure
(*c*) stresses caused by external backfills
(*d*) both (*a*) and (*b*)
(*e*) (*a*), (*b*) and (*c*) all.

Q. 487. When a gate valve, installed in a pressure pipe, is suddenly closed, water hammer pressure is caused on the pipe shell, in the portion :
(*a*) downstream of the valve (*b*) upstream of the valve
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 488. Which of the following types of pressure conduits is preferably used for large heads?
(*a*) P.C.C. pipes (*b*) Cast iron pipes
(*c*) Prestressed concrete pipes (*d*) Asbestos pipes.

CHAPTER 24

Q. 489. A hydropower plant, developed at the site of a drop in an irrigation canal, is of the type :
(*a*) runoff river plant (*b*) storage plant
(*c*) pumped storage plant (*d*) tidal plant.

Q. 490. A nuclear power development scheme is essentially a :
(*a*) thermal power scheme (*b*) hydropower scheme
(*c*) neither (*a*) nor (*b*) (*d*) both (*a*) as well as (*b*).

Q. 491. Fossil fuels are used in :
(*a*) a nuclear power plant (*b*) an ordinary thermal power plant
(*c*) a hydro power plant (*d*) none of them.

Q. 492. Major hydropower generation in India is from :
(*a*) runoff river plants (*b*) storage plants
(*c*) pumped storage plants (*d*) tidal plants.

Q. 493. A run off river plant for hydro power generation is essentially a :
(*a*) high head scheme (*b*) low head scheme
(*c*) medium head scheme (*d*) any of these.

Q. 494. A storage hydro plant essentially involves :
(*a*) a barrage or a weir (*b*) a dam
(*c*) either (*a*) or (*b*) (*d*) neither (*a*) nor (*b*).

Q. 495. The natural highwater falls may be used for generation of :
(*a*) hydro-power (*b*) water-power
(*c*) both (*a*) and (*b*) (*d*) neither (*a*) nor (*b*).

Q. 496. The power plant among the following, which is a storage plant, is :
(*a*) Rance power station in France, located on sea
(*b*) Ganguwal power house, located on Nangal hydel channel in India
(*c*) Sarda power house, located on Sarda Canal in U.P. in India.
(*d*) none of these.

Q. 497. The only statement, which is incorrect in regard to hydropower, is :
(*a*) the system efficiency of a hydroplant is quite high
(*b*) the installation cost of a hydroplant is very high
(*c*) the running cost of a hydro power plant is very low
(*d*) the hydraulic turbines takes a lot of time in being put off and on.

Q. 498. Identify the correct statement in regard to hydropower :
(*a*) hydropower stations are generally labour oriented
(*b*) Gestation period for hydro-power plant is usually small
(*c*) the hydro generators give high efficiency over a wide range of load
(*d*) in a hydropower scheme, the firm power is usually high, as compared to total power.

Q. 499. The minimum power, which a hydro-power plant can generate throughout the year, is called :
(*a*) power plant capacity (*b*) power plant load
(*c*) firm power (*d*) water power.

Q. 500. 400 cumecs of water is being released from a dam storage to meet the downstream demand, through the turbines of the connected hydroplant. The effective head of water acting on the turbines is 50 m. The efficiency of the hydroplant is 0.8. The electrical power generated from this plant, would be:
(*a*) 1,56,800 MW (*b*) 156.8 MkW
(*c*) 156.8 MW (*d*) none of these.

Q. 501. If the peak load for a power plant equals the plant capacity, then the ratio of the capacity factor to load factor will be :
(*a*) 1 (*b*) 0
(*c*) <1 (*d*) >1
(*e*) – 1 (*f*) none of these.

Q. 502*. If the peak load on a power plant having a capacity of 100 MW is 70 MW during a given week, and the energy produced is 58,80,000 kwh, the capacity factor for the plant for the week, will be :
(*a*) 35% (*b*) 50%
(*c*) 70% (*d*) none of these.

Q. 503*. The load factor for the data given in the above question will be :
(*a*) 0.35 (*b*) 0.50
(*c*) 0.70 (*d*) none of these.

Q. 504*. The utilisation factor for the data given in Q. 502, will be :
(*a*) 0.35 (*b*) 0.50
(*c*) 0.70 (*d*) none of these.

Q. 505. Pondage requirement in a hydropower plant includes the need :
(*a*) to balance the varying demand
(*b*) to compensate for wastage and spillage
(*c*) to balance short time fluctuations in the flow
(*d*) all of these.

Q. 506. A Pelton's turbine is a :
(*a*) velocity turbine (*b*) reaction turbine
(*c*) pressure turbine (*d*) none of these.

Q. 507. An impulse turbine, like a pelton's wheel, is used for :
(*a*) low heads and high discharges
(*b*) high heads and low discharges
(*c*) low heads and low discharges
(*d*) high heads and high discharges.

Q. 508. An impulse turbine, like a Pelton's turbine, is :
(*a*) an open body exposed to atmosphere
(*b*) a closed body not exposed to atmosphere
(*c*) a partially open and partially closed body
(*d*) none of these.

Q. 509. In a reaction turbine, like a Francis turbine or Kaplan turbine,
(*a*) the potential head of water is totally changed into kinetic head
(*b*) the potential head of water is partially changed into kinetic head
(*c*) the potential head of water is not at all changed into kinetic head
(*d*) all of the above are possible.

Q. 510. In a Francis turbine, the water
(*a*) flows out openly
(*b*) flows out through a closed draft tube
(*c*) both (*a*) and (*b*), depending upon the type
(*d*) neither (*a*) nor (*b*).

Q. 511*. You have to select turbines for a hydropower plant, having 150 m head of stored water behind the dam. The water in the reservoir is clear, and load on the power house is not likely to fluctuate much. Which type of turbines will you generally recommend?
(*a*) Pelton's turbines (*b*) Francis turbines
(*c*) Kaplan turbines (*d*) Any of them.

Q. 512. You have to select turbines for a hydropower plant, working on 150 m head. The water is sandy and load on the plant is highly variable. Which type of turbines will you generally recommend ?
(*a*) Pelton's turbines (*b*) Francis turbines
(*c*) Kaplan turbines (*d*) Any of them will do.

Q. 513. If two identical pumps, each capable to deliver a discharge Q against a head H, are connected :
(*a*) in parallel, the resulting discharge is $2Q$ against a head H
(*b*) in parallel, the resulting discharge is Q against a head of $2H$
(*c*) in series, the resulting discharge is $2Q$ against a head H
(*d*) in series, the resulting discharge is Q against a head $2H$.

CHAPTER 25

Q. 514. The minimum depth of water required for safe and economical navigation is:
(*a*) 1 m (*b*) 2.0 m
(*c*) 2.7 m (*d*) 4.2 m

Q. 515 The transit time taken by a barge tow, moving upstream in a navigable river, increases, if :
(*a*) the flow velocity is more (*b*) the flow velocity is less.

Q. 516. Inorder to improve the navigability in a river, the cutoffs are :
(*a*) encouraged (*b*) discouraged.

Q. 517. The belt conveyor forming an essential part of a ladder dredger, used for excavating river sand deposits, is used for :
(*a*) excavation of sand
(*b*) transporting excavated sand to the surface, over small distances
(*c*) transporting excavated sand to the surface, over large distances
(*d*) none of these.

Q. 518. The dredger which excavates sand deposits of a river, and discharges the excavated material with water over long distances, is called a :
(*a*) ladder dredger (*b*) suction dredger
(*c*) dipper dredger (*d*) none of these.

Q. 519. Lock and dam constructions for creation of a series of slack water pools along a river, for promoting its navigability, is adopted only when :
(*a*) the river is perennial
(*b*) the river carries heavy sediment load
(*c*) the available water in the river is quite less
(*d*) none of the above.

Q. 520. Timber is generally transported in our country through :

(*a*) rail routes (*b*) road routes
(*c*) air routes (*d*) water routes.

Q. 521. The oldest and cheapest mode of transporting cargo is by :

(*a*) rail routes (*b*) road routes
(*c*) air routes (*d*) water routes.

Q. 522. Ships or Steamers can ply to carry cargo, in Indian waterways, extending to about :

(*a*) 500 km (*b*) 2500 km
(*c*) 10,000 km (*d*) 40,000 km.

CHAPTER 26

Q. 523. The depth of water stored in an irrigation tank, rarely exceeds :

(*a*) 4 m (*b*) 8 m
(*c*) 12 m (*d*) none of them.

Q. 524. The storage created on the upstream side of a smaller stream by construction of low earthen bunds, is technically known as :

(*a*) a reservoir (*b*) a tank
(*c*) a lake (*d*) none of these.

Q. 525. The usual arrangement made in a tank-bund scheme to spill the surplus excess water, is known as a :

(*a*) tank spillway (*b*) tank sluice
(*c*) tank weir (*d*) none of these.

Q. 526. The provision of suitable breaching sections become more important in irrigation tanks, which are :

(*a*) isolated (*b*) in groups
(*c*) both (*a*) and (*b*) (*d*) none of them.

Q. 527. The cubic metres of water that can be stored in an irrigation tank between FTL (Full Tank Level) and Sill level of the lowest supply sluice, is known as:

(*a*) the gross capacity (*b*) effective capacity
(*c*) the specific capacity (*d*) none of the above.

Q. 528. If A_1 represents the area of a tank bed at bottom, and A_2 at FTL, then the storage capacity of this tank of height H, is given as :

(*a*) $H\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$ (*b*) $\frac{H}{2}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$

(*c*) $\frac{H}{3}\left[A_1 + A_2 + \sqrt{A_1 A_2}\right]$ (*d*) none of them.

Q. 529. The top level of a tank weir in a tank bund scheme, is kept at :

(*a*) MTL (*b*) FTL
(*c*) either (*a*) or (*b*) (*d*) none of these.

Q. 530. The capacity of an irrigation tank is sometimes increased by installing a temporary stone wall over the top of the tank weir. This fixture is known is :

(*a*) saddle (*b*) dam stones
(*c*) breast wall (*d*) divide wall.

Q. 531. The discharge through the sluice of a small irrigation tank is usually controlled by a :
(*a*) dam stone (*b*) plug
(*c*) shutter gate (*d*) none of these.

Q. 532. The mostly adopted earthen section for tank bunds is of the type :
(*a*) homogeneous embankment type
(*b*) zoned embankment type
(*c*) diaphragm type
(*d*) none of the above.

Q. 533*. The discharging capacity of a masonry waste weir of 50 m length and 1 m width provided in an earthen bund, for passing a flood with 0.8 m water depth over its crest, will be :
(*a*) 12.25 cumecs (*b*) 25.4 cumecs
(*c*) 59.4 cumecs (*d*) 65.8 cumecs.

CHAPTER 27

Q. 534. An arch dam, looks like a single arch, in :
(*a*) plan (*b*) front elevation
(*c*) side elevation (*d*) none of them.

Q. 535. An arch dam behaves as :
(*a*) a cantilever vertical retaining wall, standing up from its base
(*b*) an arch, transferring loads at the two ends by horizontal arch action
(*c*) both (*a*) and (*b*)
(*d*) none of the above.

Q. 536. A non-vertical arch dam is known as a :
(*a*) buttress dam (*b*) double curvature arch dam
(*c*) shell arch dam (*d*) both (*b*) and (*c*) above.

Q. 537. Arch dams are suitable in locations where the canyon slope factor (length of arch at top/height of dam) is :
(*a*) less than 8 (*b*) greater than 16
(U.P.S.C. Engg. Services)

Q. 538. The most economical type of arch dam in general is of :
(*a*) constant radius type (*b*) variable radius type
(*c*) constant angle type (*d*) none of these.

Q. 539. A constant angle arch dam, when compared to constant radius arch dam, utilises concrete quantity of about :
(*a*) 43% (*b*) 130%
(*c*) 230% (*d*) none of them.

Q. 540. Amongst the following types of dam sections, the thinnest and most economical section is expected for a :
(*a*) Constant angle arch dam (*b*) shell-arch dam
(*c*) constant radius arch dam (*d*) concrete gravity dam.

Q. 541. Which of the following forces is the least important in the design of arch dams?
(*a*) reservoir water force (*b*) uplift pressure
(*c*) temperature stresses (*d*) ice load
(*e*) yield stresses.

Q. 542. A constant radius arch dam is also sometimes called a :
(*a*) constant centre arch dam (*b*) constant angle arch dam
(*c*) variable angle arch dam (*d*) none of the above.

Q. 543. The most economical central angle of the arch rings in an arch dam, is :
(*a*) 123°—34′ (*b*) 133°—34′
(*c*) 143°—34′ (*d*) 153°—34′.

Q. 544. The most economical central angle of the arch rings of an arch dam, can be adopted only at one place, preferably at mid height in an arch dam of the type :
(*a*) constant angle arch dam (*b*) constant radius arch dam
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 545. In an arch dam, the 'extrodos curves' refer to the arch rings corresponding to the :
(*a*) upstream face of the dam
(*b*) downstream face of the dam
(*c*) either (*a*) or (*b*)
(*d*) none of the above.

Q. 546. A V-shaped valley with stronger foundations can suggest the choice of an arch dam of the type :
(*a*) constant radius arch dam
(*b*) variable radius arch dam
(*c*) constant angle arch dam
(*d*) none of the above.

Q. 547. The type of an arch dam, which generally requires 'overhangs at abutments', is of :
(*a*) constant radius type (*b*) variable radius type
(*c*) constant angle type (*d*) none of the above.

Q. 548. The most accurate method to design arch dams is based upon :
(*a*) the thin cylinder theory (*b*) the theory of elastic arches
(*c*) the trial load method (*d*) all of the above.

Q. 549. Which among the following is not an assumption of the 'Trial Load Analysis' method of design of arch dams ?
(*a*) plane sections normal to the axis remain plane after flexure
(*b*) any horizontal arch ring is independent of the arch rings below and above
(*c*) modulus of elasticity of concrete and rock foundation is equal for compression and tension
(*d*) stresses are proportional to strains.

Q. 550. The thin cylinder theory for designing arch dams, ignores :
(*a*) temperature stresses (*b*) ice pressures
(*c*) yield stresses (*d*) all of the above.

Q. 551. The only force considered in the design of arch dams by thin cylinder theory is :
(*a*) water pressure (*b*) yield stresses
(*c*) shrinkage stresses (*d*) none of them.

Q. 552. The temperature stresses, producing worst effects in the design of arch dams, are caused by :
(*a*) rise of temperatures (*b*) fall of temperatures
(*c*) both (*a*) and (*b*) (*d*) none of the above.

Q. 553. The stresses due to rib-shortening become quite important in the case of :
(*a*) long thin arch dams (*b*) thick arch dams of small angle
(*c*) both (*a*) and (*b*) above (*d*) none of the above.

Q. 554. While designing a freely supported deck type of a buttress dam, the chosen spacing between buttresses is usually increased to achieve economy, from about 4.5 to 15 m or so, with the :
(*a*) decrease in dam height from about 45 m to about 15 m
(*b*) increase in dam height from about 15 m to about 45 m.

Q. 555. The load from the deck of a buttress dam is transmitted to the foundations, through the :
(*a*) corbels (*b*) buttresses
(*c*) lateral braces (*d*) none of them.

Q. 556. Which of the following components in a buttress dam are responsible for transmission of load from the sloping deck slab to the buttresses?
(*a*) corbels (*b*) lateral braces
(*c*) counterforts (*d*) none of them.

Q. 557. The most suitable foundation for buttress dams is :
(*a*) pile foundation (*b*) well foundation
(*c*) mat foundation (*d*) none of them.

Q. 558. The lateral braces are provided between buttresses in a buttress dam, to :
(*a*) transfer the load from the deck to the foundation
(*b*) transfer the load from the deck to the buttresses
(*c*) provide resistance against buckling of buttresses
(*d*) none of the above.

Q. 559. No reinforcement is necessary in the :
(*a*) fixed deck buttress dam
(*b*) simple deck buttress dam
(*c*) cantilever deck buttress dam
(*d*) massive head buttress dam.

Q. 560. Which of the following dam is also known as Ambursen dam?
(*a*) multiple arch buttress dam
(*b*) mushroom head buttress dam
(*c*) massive head buttress dam
(*d*) free deck buttress dam.

Q. 561. Which of the following component of a buttress dam is also called a counterfort?
(*a*) corbel (*b*) buttress
(*c*) lateral brace (*d*) sloping deck.

Q. 562. Which of the following is incorrect about buttress dam?
(*a*) it requires less construction material than a solid gravity dam of the same height
(*b*) it is less susceptible to deliberate damage or sabotage
(*c*) Ice pressure is not significant, because ice slides over the sloping deck
(*d*) the factor of safety is usually greater than for a gravity dam.

Q. 563. Multiple dome buttress dam is a type of :
(*a*) rigid buttress dam (*b*) deck slab buttress dam
(*c*) bulk head buttress dam (*d*) none of the above.

Q. 564. In a simple deck slab buttress dam of a given height, the buttress spacing can be increased by :

(*a*) increasing the upstream slope
(*b*) decreasing the upstream slope
(*c*) either (*a*) or (*b*)
(*d*) none of these.

Q. 565. The buttresses of a free deck buttress dam are provided @ 'x' m clear spacing, and buttress thickness is proposed to be 't' m. The buttress section can then be designed like the section of a concrete gravity dam, by considering the effective unit weight of water = surcharge factor(S) × actual unit weight of water (γ_w). The value of this surcharge factor is given as :

(*a*) $S = \dfrac{x+t}{x}$ (*b*) $S = \dfrac{x+t}{t}$

(*c*) $S = \left(\dfrac{x+t}{1+t}\right)$ (*d*) none of these.

Q. 566. Multiple arch dams are generally used for :

(*a*) high heights (*b*) low heights
(*c*) medium heights (*d*) all of the above.

ANSWERS OF THE OBJECTIVE QUESTIONS

1. (*d*)	2. (*a*)	3. (*d*)	4. (*a*)	5. (*c*)
6. (*a*)	7. (*b*)	8. (*b*)	9. (*d*)	10. (*a*)
11. (*b*)	12. (*c*)	13. (*a*)	14. (*b*)	15. (*b*)
16. (*b*)	17. (*b*)	18. (*d*)	19. (*b*)	20. (*c*)
21. (*a*)	22. (*c*)	23. (*a*)	24. (*c*)	25. (*b*)
26. (*b*)	27. (*d*)	28. (*b*)	29. (*a*)	30. (*c*)
31. (*c*)	32. (*a*)	33. (*c*)	34. (*b*)	35. (*b*)
36. (*b*)	37. (*b*)	38. (*b*)	39. (*c*)	40. (*a*)
41. (*c*)	42. (*b*)	43. (*c*)	44. (*c*)	45. (*b*)
46. (*c*)	47. (*c*)	48. (*c*)	49. (*b*)	50. (*a*)
51. (*d*)	52. (*d*)	53. (*c*)	54. (*c*)	55. (*c*)
56. (*b*)	57. (*d*)	58. (*d*)	59. (*a*)	60. (*a*)
61. (*b*)	62. (*c*)	63. (*b*)	64. (*d*)	65. (*a*)
66. (*c*)	67. (*d*)	68. (*c*)	69. (*b*)	70. (*c*)
71. (*c*)	72. (*a*)	73. (*a*)	74. (*b*)	75. (*d*)
76. (*b*)	77. (*b*)	78. (*d*)	79. (*b*)	80. (*d*)
81. (*c*)	82. (*c*)	83. (*b*)	84. (*a*)	85. (*b*)
86. (*c*)	87. (*a*)	88. (*c*)	89. (*c*)	90. (*d*)
91. (*d*)	92. (*c*)	93. (*a*)	94. (*a*)	95. (*c*)
96. (*c*)	97. (*b*)	98. (*d*)	99. (*b*)	100. (*c*)
101. (*c*)	102. (*b*)	103. (*b*)	104. (*c*)	105. (*b*)
106. (*a*)	107. (*b*)	108. (*a*)	109. (*b*)	110. (*d*)
111. (*b*)	112. (*b*)	113. (*b*)	114. (*b*)	115. (*b*)
116. (*a*)	117. (*c*)	118. (*c*)	119. (*a*)	120. (*c*)
121. (*b*)	122. (*b*)	123. (*c*)	124. (*c*)	125. (*c*)
126. (*a*)	127. (*b*)	128. (*c*)	129. (*c*)	130. (*d*)
131. (*a*)	132. (*a*)	133. (*c*)	134. (*b*)	135. (*c*)

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566. (*a*).

HINTS FOR SOLVING STARRED QUESTIONS

Q. 16. *Follow example 1.1, and compute :*

From eqn (1.1)

$$T = 2.3 \cdot \frac{y}{f} \log_{10}\left(\frac{Q}{Q - fA}\right)$$

where $A = 0.203$ ha $= 2030$ m^2

$Q = 0.043$ cumecs $= 0.043 \times 60 \times 60$ m^3/h

$= 154.8$ m^3/h $= 5$ cm/hr $= 0.05$ m/h

$y = 6.35$ cm $= 0.0635$ m

$$T = 2.3 \times \frac{0.0635}{0.05} \log\left(\frac{154.8}{154.8 - 0.05 \times 2030}\right) = \mathbf{1.35\ h.} \quad \textbf{Ans.}$$

Q. 17. Combining wt $= \dfrac{\text{Atomic wt}}{\text{Valency}}$

$\therefore$ Combining wt of Na $= \dfrac{23}{1} = 23$

Combining wt of Ca $= \dfrac{40}{2} = 20$

Combining wt of Mg $= \dfrac{24}{2} = 12$

$$Na^{+} = \frac{\text{Na in mg/l}}{23} = \frac{345}{23} = 15 \text{ milli equivalent per litre}$$

$$Ca^{++} = \frac{60}{20} = 3 \text{ milli equivalent per litre}$$

$$Mg^{++} = \frac{18}{12} = 1.5 \text{ milli equivalent per litre}$$

From eqn. (1.4) :

$$SAR = \frac{Na^{+}}{\sqrt{\dfrac{Ca^{++} + Mg^{++}}{2}}} = \frac{15}{\sqrt{\dfrac{3 + 1.5}{2}}} = \frac{15}{1.5} = \mathbf{10.} \quad \textbf{Ans.}$$

Q. 45. Specific weight means unit weight $= 14.7$ kN/m^3.

Also, unit wt of water $= 9.8$ kN/m^3:

Available moisture holding capacity of root zone

$$= \frac{\gamma d}{\gamma_w}\left(\text{F.C. m.c.} - \text{Wilting pt m.c.}\right)$$

$$= \left(\frac{14.7}{9.8}\right) \times (0.9\ m)\left[25\% - 15\%\right]$$

$$= 1.5 \times (0.9 \text{ m}) \times 10\% = 0.135 \text{ m} = \mathbf{13.5\ cm.} \quad \textbf{Ans.}$$

Q.47. Water depth required at canal

$$= \frac{\text{Water depth required in the field}}{\eta_a \cdot \eta_c}$$

$$= \frac{10 \text{ cm}}{0.8 \times 0.9} = 13.9 \text{ cm}$$

$\therefore$ Volume of water required for 10 hectare (10×10^4 m^2) field

$$= \frac{13.9}{100} \text{ m} \times 10^5 \text{ m}^2 = 13{,}900 \text{ m}^3 = \mathbf{13{,}900 \text{ kl.}} \quad \textbf{Ans.}$$

Q. 48. **Storage duty** or **Quantity duty** is the crop hectares, which can be fully grown/irrigated by 1 M. cum of water.

Thus, if Δ (m) is the water depth required by the crop for its maturity, then D hectares of crop will evidently require $(D \times 10^4)\,\Delta$ m^3 of water, which is 1 M. cum.

$\therefore \quad (D \times 10^4\,\Delta \text{ m}^3 = 1 \text{ M cum} = 10^6 \text{ m}^3$

or $\quad D \times \Delta = 100$

or $\quad \Delta = \dfrac{100}{D}$

where Δ is in m

D is in ha/M. cum.

Q. 55. I.I. for Kharif $= 100 - 72 = 28\%$

I.I. for Rabi $= 100 - 56 = 44\%$

Total I.I. $= 28 + 44 = \mathbf{72\%}$. **Ans.**

Q. 57. Such an area may be called as a seasonal cropped area, or seasonal sown area, such as *Rabi area or Kharif area.* Net sown area is the total cropped area (which is only cropped once) in the entire year, *i.e.* in both the seasons.

Q. 58. $G.I.A = 4.5 \times 0.9 + 2.5 \times 0.8 = 6.05$ M ha

$$A.I.I = \frac{G.I.A.}{CCA} = \frac{6.05}{5} = \mathbf{1.21\ i.e.\ 121\%}. \quad \textbf{Ans.}$$

Q. 66. Required discharge (mean) in the field

$= 0.4$ cumecs.

Peak discharge required

$$= \frac{\text{Mean discharge}}{\text{Capacity factor}} \qquad \left(\because C.F. = \frac{Q_{mean}}{Q_{peak}}\right)$$

$$= \frac{0.4}{0.8} = 0.5 \text{ cumecs}$$

Design discharge for distributary

$$= \frac{\text{Required discharge for crops}}{\text{Time factor}}$$

(Since channel runs for lesser days than the crop days, the required channel discharge should be more)

$$= \frac{0.5}{0.5} = \mathbf{1 \text{ cumecs.}} \quad \textbf{Ans.}$$

Q. 67. Discharge required at head of canal

$$= \frac{6.25 \text{ cumecs}}{0.75} = 8.33 \text{ cumecs}$$

Area irrigated $= 5100$ hectares

Duty on capacity

$$= \frac{\text{Area to be irrigated during base period}}{\text{Design full supply discharge at its head during peak demand}}$$

$$= \frac{5100}{8.33} = \textbf{612 ha/cumecs.} \quad \textbf{Ans.}$$

Q. 69. $G.C.A = 8000$ ha

$CCA = GCA -$ Reserved forests, etc. $= 8000 - 1600 = 6400$ ha

Pastures and fallow lands are included in CCA, and hence not to be subtracted.

Area under irrigation

$= CCA \times$ In. of Irr. $= 6400 \times 0.5 = 3200$ hectares. **Ans.**

Q. 79. $$\frac{\tau_0}{\gamma \cdot d\,(S_s - 1)} = 0.056$$

or $$\tau_\theta \propto d. \quad \textbf{Ans.}$$

Q. 80. For non-scouring

$$d < 11\,RS$$

or $$R_{max} = \frac{d}{11S} \qquad ...(i)$$

$$\therefore \quad V = \frac{1}{N} \cdot R^{2/3} \cdot S^{1/2}$$

But $$N = \frac{1}{24} d^{1/6}$$

$$\therefore \quad V = \frac{24}{d^{1/6}} \cdot R^{2/3} \cdot S^{1/2}$$

$$V_{max} = \frac{24}{d^{1/6}} \cdot R_{max}^{2/3} \cdot S^{1/2}$$

$$= \frac{24}{d^{1/6}} \cdot \left(\frac{d}{11S}\right)^{2/3} \cdot S^{1/2}$$

$$= \frac{24}{(11)^{2/3}} \cdot d^{2/3 - 1/6} \cdot S^{1/2 - 2/3}$$

$$= 4.85\, d^{1/2} \cdot S^{-1/6}$$ where d is in m

When d is in cm *i.e.* d_{cm}, then

$$d = \frac{d_{cm}}{100}$$

$$\therefore \quad V_{max} = 4.85 \left(\frac{d\,cm}{100}\right)^{1/2} \cdot S^{-1/6}$$

$$= 0.485\, d^{1/2} \cdot S^{-1/6}$$ where d is in cm

or $$V_{max} = 0.48\, d^{1/2} \cdot S^{-1/6}. \quad \textbf{Ans.}$$

Q. 81. N is given by Strickler's formula, as

$$N = \frac{1}{24} \cdot d^{1/6}$$

where d is representative grain dia in m.

Q. 85. Such channels are designed only for non-scouring. Hence, they must carry clear waters only. **Ans.**

Q. 86. For 1.5 H : 1 V,

$$\tan\theta = \frac{y}{x} = \frac{1}{1.5}$$

$$\sin\theta = 0.554$$

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \left(\frac{0.554}{\sin 37°}\right)^2}$$

$$= \sqrt{0.153} = 0.391.$$

For stability on banks

$$0.75\,\gamma RS \le 0.391\,\tau_c$$

$$\le 0.391\,(\gamma RS)$$

For stability on bed,

$$RS = \frac{d}{11}$$

$$0.75\,\gamma \cdot RS \le 0.391 \cdot \gamma \cdot \frac{d}{11}$$

or $$RS \le \frac{0.391}{11 \times 0.75} d$$

or $$RS \le \frac{d}{21}$$

or $$R \le \frac{d}{21S}.$$ **Ans.**

Q. 89. If the actual velocity in the channel exceeds the optimum velocity (V_0), then scouring may occur ; and if the actual velocity is lesser than the optimum velocity (V_0), then silting may occur. This optimum velocity, thus, represents both maximum as well as minimum velocity.

Q. 104 & Q. 108. When a canal has a *fixed configuration* (*i.e.* its bed slope and cross-section do not change with time) due to its non-erodible surface, such as in a lined canal, then the channel is called in **permanent regime.** Besides no scouring, usually, there is no silting in such channels, because higher velocities are permitted, due to their inerodible surface. Hence, the silt amount leaving the channel is almost the same as the silt amount entering the channel. Such channels, whose boundaries are fixed and rigid, are called **rigid boundary canals,** and Lacey's theory is not applicable to them. *The resistance offered to the flowing water in such a channel, is a function of the nature of the boundary surface alone.* On the other hand, when the canals are constructed in erodible strata, their boundaries tend to change continuously with the flow of water in them. Such channels are evidently called **mobile boundary canals.** The material forming the boundaries of such channels are mostly loose soils, easily susceptible to erosion by flowing water. In such canals, higher velocities are usually not permissible, because this would lead to excessive scouring. Lower values of velocities will not be able to keep the entire silt load into suspension, which may lead to silting in them. Due to such scouring and silting of sediment, the sediment load transport at different sections along the channel, will usually be different. Silting and scouring may also result in modification of channel

cross-section. *The resistance offered to the flow of water in such channels depends on two factors. viz :*

(i) nature of the surface of boundaries, and

(ii) condition of the bed and banks of the channel.

Such channels need be designed for no-nsilting and non-scouring velocity. Rivers and unlined alluvial canals are examples of mobile boundary channels.

Q. 109. Scour depth $= 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{3^2}{1.2}\right)^{1/3} = 2.64$ m. **Ans.**

Q. 111. From eqn. (4.14) :

$$f = 1.75\sqrt{d_{mm}} = 1.75\sqrt{0.4} = 1.107$$

From eqn. (4.19) :

Regime bed slope $= \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{(1.107)^{5/3}}{3340(40)^{1/6}} = \frac{1}{5200}$. **Ans.**

Q. 112.

$$1.0 = 1.75\sqrt{d_{mm}}$$

$$d_{mm} = \left(\frac{1}{1.75}\right)^2 = 0.326 \text{ mm} \approx 0.32 \text{ mm}. \quad \textbf{Ans.}$$

Q. 114.

$$P = b + \sqrt{5}\cdot y;$$

with $\frac{1}{2}$ H : 1 V slopes, which are generally attained in a regime canal, irrespective of Q and f

$$= 22 + \sqrt{5} \times 2.5 = 27.59 \text{ m}$$

$$P = 4.75\sqrt{Q} = 27.59$$

$\therefore$

$$Q = \left(\frac{27.59}{4.75}\right)^2 = 33.73 \text{ m}^3/\text{s} \approx 34 \text{ cumecs}. \quad \textbf{Ans.}$$

Q. 115. Given $\begin{cases} Q_A = Q_B \\ f_A > f_B \end{cases}$

Now since, $V = \left(\frac{Qf^2}{140}\right)^{1/6}$

and $P = 4.75\sqrt{Q}$

$V_A > V_B$

$P_A = P_B$

Since Q is same, $A_A < A_B$

and since $R = \frac{A}{P}$

$R_A < R_B$ or $R_B > R_A$

i.e. channel B has more hydraulic radius for the same wetted perimeter, and this happens only when the channel has more depth (deeper channels are more hydraulically efficient).

$\therefore$ $y_B > y_A$.

Hence, choice (*b*) is the correct choice. **Ans.**

Q. 122. By the geometry of Fig. 29.1, it can be seen that the points, B, B', etc. will lie on the bank slope line, only when the berm equals $(n_2 - n_1)\, d_1$,

where d_1 is the depth of cutting, which will vary according to ground level.

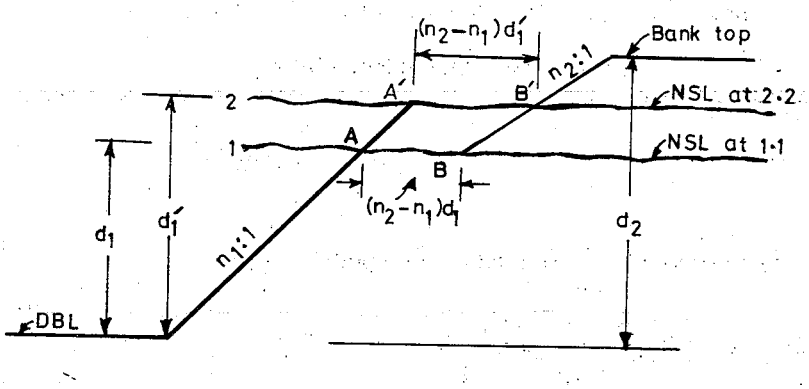


Fig. 29.1

Q. 123. It can be seen from Fig. 29.2 that when channel attains regime, $\frac{1}{2}$ H : 1 V side

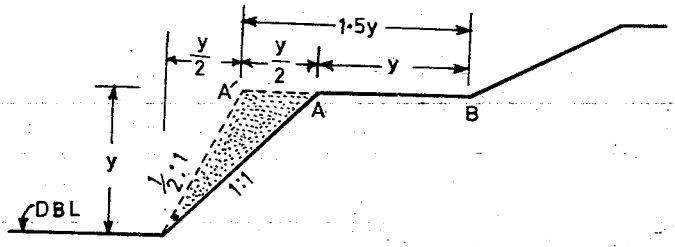


Fig. 29..2

slopes will be formed, thereby increasing the initial berm width AB to $A'B$, where

$$A'B = A'A + AB = 0.5\,y + y = 1.5\,y. \quad \textbf{Ans.}$$

Q. 126. *Assume 1 km length*

$$\text{Annual benefit} = 1000\text{ m} \times 20\text{ m} \times \frac{3}{10^6}\text{ cumecs} \times \text{Rs. 4 lakh/yr cumecs}$$

$$= \text{Rs. } 24000 \text{ per yr}$$

$$\text{Annual cost} = \frac{100\text{ m} \times 20\text{ m}}{40\text{ yr}} \times \text{Rs. } 40/\text{m}^2 = \text{Rs. } 20{,}000 \text{ per yr.}$$

$$\text{B.C. ratio} = \frac{24000}{20000} = 1.2. \quad \textbf{Ans.}$$

Q. 129. W.r.t. Fig. 29.3 ; $\tan\theta = \frac{1}{1.5}$

$\theta = 0.588$

$A = y^2(\theta + \cot\theta)$

$= 2.5^2 (0.588 + 1.5) = 13.05\ m^2$

$V = \frac{26}{13.00} \cong 2$ m/s. **Ans.**

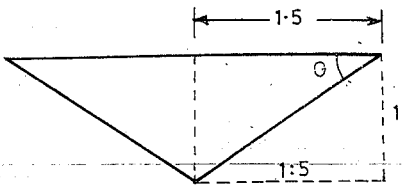


Fig. 29.3

Q. 147. The classification of saline and alkaline soils is as follows :

Sl. No.	*Classification*	*Electrical Conductivity (EC) in micro-mho/cm*	*Exchangeable Sodium Percentage (ESP)*	*pH value*
1.	**Saline soil** or *white alkali*	>4000	< 15	≤ 8.5
2.	**Alkaline soil** or *Non-saline alkali* or *sodic soil* or *black alkali*	< 4000	> 15	8.5 to 10.0
3.	**Saline-alkali soil**	> 4000	> 15	< 8.5

Q. 166. $M_B \approx 150 \cdot \sqrt{Q_a} = 150\sqrt{1600} = 6000$ m *i.e.* 6 km. **Ans.**

Q. 169. $M_B = 3$ km $= 3000$ m

Also, $M_B \approx 150 \cdot \sqrt{Q_{dominant}}$

$\therefore \quad 3000 = 150\sqrt{Q_{dominant}}$

or $Q_{dominant} = 400$ cumecs

$\therefore \quad Q_{max} = \frac{16}{9} \times 400 = 711$ cumecs ≈ 700 cumecs. **Ans.**

Q. 176. Average thickness of apron

$= T = 1.9t = 1.9 \times 0.06\,(5000)^{1/3}$

$= 1.95$ m ; say 2 m. **Ans.**

Average width of apron

$= 1.5\,D$

where $D = 1.5\,R - y$ (average), where

$$R = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

$= 0.47\,(5000)^{1/3}$ (assume $f = 1$) $= 8.04$

$= 1.5 \times (1.5 \times 8.04 - 5)$

$= 10.6$ m ≈ 10 m (say). **Ans.**

Q. 178. Waterway required

$= 1.2 \times 4.75 \cdot \sqrt{Q_{max}}$

where 1.2 is safety factor of Lacey's regime width

$$= 1.2 \times 4.75 \times \sqrt{1600 \times \frac{16}{9}}$$

$$\left(\text{since } Q_{max} = \frac{16}{9} \cdot Q_{dom} \right)$$

$$= 304 \text{ m} \approx 300 \text{ m}. \quad \textbf{Ans.}$$

Q. 216. Only nominal thickness is required under upstream floor.

Q. 219. ∵ Fine sand will get lifted most easily.

Q. 297. Refer and study article 14.5.6.

Q. 298. Refer and study article 14.5.6.

Q. 304.
$$R = 1.35 \left(\frac{q^2}{f} \right)^{1/3} = \left(\frac{2500}{300} \right)^{2/3} \qquad \text{(assume } f = 1\text{)}$$
$$= 5.55 \text{ m}$$

Foundation may be taken down below *HFL* by $1.33R$, or $R + 1.2$ m, whichever is more.

(*i*) $1.33\,R = 7.38$ m

(*ii*) $R + 1.2 \text{ m} = 5.55 + 1.2 = 6.75 \text{ m} < 7.38 \text{ m}$

Hence, foundation level

$= 214 - 7.38 = 206.62$ m

say 206.6 m. **Ans.**

Q. 323 to Q. 325. **Aquifuge** is that geological formation, which is neither porous nor permeable ; and hence it neither contains nor yields ground water. *Granite rock is an example of aquifuge.*

Aquitard is that geological formation, which does not yield water freely to wells due to its lesser permeability, although seepage is possible through it. The yield from such a formation is, thus, insignificant. *Sandy clay is an example of aquitard.*

Aquiclude is highly porous, containing large quantities of water, but essentially impervious, as not to yield water. *A clay layer is an example of aquiclude.*

Q. 336. The permeability coefficient, K, is given by the eqn.

$$K = C \cdot d^2 \frac{\gamma_w}{\mu} = \frac{C \cdot d^2 \rho_w g}{\rho \cdot \nu}$$

or
$$K = \frac{C d^2 \cdot g}{\nu} \qquad \ldots(29.1)$$

where C = constant called **shape factor**, depending on porosity, packing and grain size distribution of the porous medium

d = mean particle size of porous medium

γ_w = unit weight of water flowing through soil

μ = dynamic viscosity of water.

Thus
$$K \propto \frac{1}{\nu}$$

ν decreases with increase in temperature (Table 1 at the end of the book), and hence K will increase with increase in temperature. **Ans.**

Q. 337. From Eq. (30.1)

$$K = C \cdot d^2 \frac{g}{\nu}$$

where ν represents the property of fluid passing through the porous medium

C and d represents the properties of porous medium only.

The component of permeability which represents only the properties of the porous media, is called **intrinsic or specific permeability,** and represented by K_0,

$$\text{where } K_0 = C \cdot d^2$$

Its units are m^2 or darcy,

$$\text{where 1 darcy} = 9.87 \times 10^{-13}\ \text{m}^2.$$

The coefficient K_0 is generally used, when water/fluid temperature varies considerably or where more than one fluid is involved. However, in ground water flow, the temperature variation is small, and only water is involved as the flowing fluid, only K is considered.

Q. 340. A is defined as volume of water (m^3) released from an aquifer of 1 m^2 area and full depth d, per m fall of piezometric head. Its units will, thus, be $m^3/m^2/m$; *i.e.* dimensionless.

Q. 344. Specific yield and storage coefficient are related to a given aquifer and not to a given well.

Q. 348. Since Specific yield + Specific retention = Porosity, the specific yield, evidently, is less than porosity.

Q. 370. Ramganga dam, although 128 m high, is earthen and rockfill dam. Nagarjuna Sagar dam is 125 m high, Hirakud dam is 59 m high, and Mettur dam is 70 m high ; they are all stone masonry gravity dams.

Q. 396. Dead storage $= 0.2 \times 30 = 6$ M. cum

Time during which 6 M. cum is filled by sediment

$$@0.1 \text{ M. cum/yr} = \frac{6}{0.1} = 60 \text{ years.} \quad \textbf{Ans.}$$

Q. 405. For $F > 32$ km, the wave ht is given by eqn. (19.12) as

$$h_w = 0.032 \cdot \sqrt{V \cdot F}\ \text{m}$$
$$= 0.032\sqrt{160 \times 40} = 2.56\ \text{m}$$

Force caused by waves P_w is given by eqn. (19.14) as

$$P_w = 19.62\, h_w^2\ \text{kN/m run of dam}$$
$$= 19.62 \times (2.56)^2\ \text{kN} = 128.6\ \text{kN}$$
$$\approx 130\ \text{kN}.$$ Hence, choice (d) is correct. **Ans.**

Q. 406. The formula at choice (a) is correct. This may be remembered, as some numerical may be asked on this.

Q. 407. From eqn. (19.11),

wave ht $(h_w) = 0.032\sqrt{V \cdot F} + 0.763 - 0.271\,(F)^{1/4}$ for $F < 32$ km

$$\therefore \quad h_w = 0.032 \cdot \sqrt{100 \times 20} + 0.763 - 0.271\,(20)^{1/4}$$
$$= 1.431 + 0.763 - 0.573 = 1.62 \text{ m}$$

Free-board is generally provided equal to

$$1.5\, h_w = 1.5 \times 1.62 = 2.45 \text{ m, say } 2.5 \text{ m.} \quad \textbf{Ans.}$$

Q. 414. From eqn. (19.22) :

$$B = \frac{H}{\sqrt{S_s - c}} = \frac{H}{\sqrt{2.4 - 1}} = \frac{60}{\sqrt{1.4}}$$
$$= 50.7 \text{ m (with full uplift = 1)}$$

From eqn. (19.23) :

$$B = \frac{H}{\mu\,(S_s - c)} = \frac{60}{0.7\,(1.4)} = 61.22 \text{ m; say } 64 \text{ m.}$$

Chosen B should be greater of the two values, *i.e.* 61 m. **Ans.**

Q. 431. Cut off can be provided only during construction. But if any seepage appears afterwards, a drain with inverted filter along toe line is provided to collect soil-free water, to check piping.

Q. 447. Since an ogee spillway is designed for maximum head, up to *RL* of maximum reservoir level, there are almost no chances of operating head exceeding the design head. However, under rare emergency, when extraordinary flood is being passed near the end of monsoon season, it may become necessary to allow the reservoir level to even go above the Maximum Reservoir level, by encroaching upon the freeboard, in which case, the operating head may exceed the design head.

Q. 449. The efficiency of spillway indirectly means its coefficient of discharge, which increases with the increasing head above full reservoir level (*FRL*) till it becomes about 2.2 at full design head (*i.e.* at MRL). Since the spillway starts working when water level just crosses *FRL*, the coefficient of discharge in early stages, is only about 1.7, *i.e.* about 77% of its value at full design head. Hence, the spillway efficiency can be said to be increasing from 77% to 100%, as water level in the reservoir increases from F.R.L. to M.R.L.

Q. 461. A saddle syphon is nothing but a syphon spillway, and includes both of its types. Air vent is the essential requirement of both types of syphon spillways.

Q. 475. $Q = C_d \cdot A \sqrt{2gH_1}$

where C_d = Coefficient of discharge through orifice

H_1 = Water head over the centre line of opening
$= RL331 - RL328 = 3$ m

A = Area of opening
$= 60 \text{ m} \times 6 \text{ m} = 360 \text{ m}^2$

$$\therefore \quad Q = 0.62 \times 360 \sqrt{2 \times 9.81 \times 3}$$
$$= 1712 \text{ cumecs} \approx 1700 \text{ cumecs.} \quad \textbf{Ans.}$$

Q. 476. $Q = C_d A \cdot \sqrt{2gH}$

H = Water head over centre line of sluice (since it is a free overfall on downstream side)
$= RL330\,(FRL) - RL300$ m (sluice centre line level)
$= 30$ m

$$A = \frac{\pi}{4} \times 2^2 = 3.14 \text{ m}^2$$

C_d = Coefficient of discharge ≈ 0.62 for free overfall

$$\therefore \quad Q = 0.62 \times 3.14 \cdot \sqrt{2 \times 9.81 \times 30}$$

$$= 47.23 \text{ cumecs}$$

say 50 cumecs (nearest appropriate choice given). **Ans.**

Q. 477. $$Q = C_d \cdot A \cdot \sqrt{2gH_L}$$

$$= 0.8 \times \left(\frac{\pi}{4} \times 4^2\right) \sqrt{2 \times 9.81 \times (226 - 210)}$$

$$= 0.8 \times 4\pi \sqrt{16.62 \times 16} = 164 \text{ cumecs}$$

≈ 170 cumecs (nearest appropriate choice). **Ans.**

To countercheck, let us see the absolute maximum discharge, which computes to be 204.8 cumecs at $C_d = 1$. In any case, even if all losses are neglected, Q cannot exceed 204.8 cumecs, and hence the last choice is certainly incorrect, other choices at (*a*) and (*b*) are giving very low coefficient of discharge, such as $\frac{64}{204.8} = 0.3$ and $\frac{104}{204.8} = 0.5$, which are too low for submerged bell mouthed pipe flows.

Q. 502. From eqn. [17.6 (*a*)] :

Capacity factor (C.F.)

$$= \frac{\text{Average load}}{\text{Plant capacity}}$$

$$\text{where, average load} = \frac{\text{Energy produced}}{\text{Time in h}}$$

$$= \frac{58,80,000}{7 \times 24} \text{ kW}$$

$$= 35000 \text{ kW} = 35 \text{ MW}$$

$$\text{C.F.} = \frac{35 \text{ MW}}{100 \text{ MW}} = 0.35. \quad \textbf{Ans.}$$

Q. 503. Load factor $= \frac{\text{Average load}}{\text{Peak load}} = \frac{35 \text{ MW}}{70 \text{ MW}} = 0.5.$ **Ans.**

Q. 504. Utilisation Factor (*UF*)

$$= \frac{\text{Maximum power utilised}}{\text{Maximum power available}} = \frac{70 \text{ MW}}{100 \text{ MW}} = 0.7. \quad \textbf{Ans.}$$

Q. 511. Francis turbines are cheaper than Pelton's turbines, and there is no special need for Pelton's turbines here, because part load (variable loads) are not likely to come and water is not sandy, which may certainly need Pelton's turbines. Hence, Francis turbines will suffice to our needs. **Ans.**

Q. 533. $Q = C \cdot LH^{3/2} = 1.66 \times 50 \times (0.8)^{3/2}$

($\because$ When width of rectangular crest > 0.9 m, $C = 1.66$)

$= 59.4$ cumecs. **Ans.**

30

Test Paper on Objective Questions

Q. 1. In a hot arid district of Rajasthan State, like Jodhpur, the preferred mode of irrigation, would be :

(*a*) flood irrigation (*b*) furrow irrigation

(*c*) drip irrigation (*d*) sprinkler irrigation,

Q. 2. The conjunctive use of water in a basin is defined as :

(*a*) the Evapotranspiration needs of the basin

(*b*) combined exploitation of surface and ground waters in the basin

(*c*) combined utilisation of surface waters for kharif as well as Rabi crops in the basin

(*d*) none of the above.

Q. 3. The ratio of the average value of the shear stress produced on the bed, and that on the banks of an alluvial canal, due to the action of the flowing water, is :

(*a*) 0.75 (*b*) 1.0

(*c*) 1.33 (*d*) 0.

Q. 4. Lacey's silt theory is not applicable when :

(*a*) silt grade consists of pure sand

(*b*) silt amount is of the order of 500 ppm

(*c*) the canal is lined

(*d*) none of the above.

Q. 5.* An alluvial irrigation canal has been excavated with 1 : 1 side slopes in cutting and $1\frac{1}{2}$: 1 (H : V) in filling. At a given cross-section of this canal, the cutting depth is found to be D, and water depth up to FSL as 1.2 D. The berm width left originally is D/2, which, when channel attains regime, will become :

(*a*) $\frac{D}{2}$ (*b*) D

(*c*) $1.2D$ (*d*) $1.3D$.

Q. 6.* The maximum area that can be irrigated by a stream discharge of 0.05 cumecs, if the average rate of infiltration is 5 cm/h, is :

(*a*) 0.36 hectares (*b*) 100 hectares

(*c*) 0.84 hectares (*d*) 10.26 hectares. *(Calcutta University)*

Q. 7. The ratio of the quantity of water stored in the root zone of the crops to the quantity of water actually delivered into the field, is called :

(*a*) efficiency of water distribution

(*b*) efficiency of water storage

(*c*) efficiency of water use

(*d*) efficiency of water application.

Q. 8. The hydraulic radius of a standard trapezoidal lined canal with 1.25 H : IV side slopes, 12 m bed width and 2 m FSD, is :

(*a*) 1.11 m (*b*) 1.41 m
(*c*) 1.61 m (*d*) none of them.

Q. 9. Safety ladders are provided to enable :

(*a*) the fish to move in water from downstream to upstream to their ancestoral spawning grounds
(*b*) the fish to move in water from upstream to downstream in search of food
(*c*) both (*a*) and (*b*) above
(*d*) none of the above.

Q. 10. Which one of the following statements is not true about cement concrete lining ?

(*a*) that it develops frequent cracks due to temperature changes
(*b*) that it develops frequent cracks due to settlement of subgrade
(*c*) that it is likely to be damaged by alkaline water
(*d*) that it can be easily punctured by weed growth.

Q. 11.* River yamuna in the city reach of Delhi, has to carry a maximum discharge of about 5000 cumecs. The river has been jacketed on both sides by dikes at 2 km apart. These dikes come under frequent attacks of the meandering river, and need elaborate protective anti-erosion works, because the dikes are closely spaced, curtailing the original meander belt by about :

(*a*) 10% (*b*) 25%
(*c*) 50% (*d*) 75%.

Q. 12.* The meander width for an alluvial stream is estimated to be about 3.4 km. The normal width of this stream shall roughly be of the order of :

(*a*) 100 m (*b*) 200 m
(*c*) 30 m (*d*) 600 m.

Q. 13. An Indian irrigation project designed to serve a command of more than 2000 hectares and up to 10000 hectares, is known as a :

(*a*) major irrigation project
(*b*) minor irrigation project
(*c*) medium irrigation project
(*d*) none of them, since irrigation projects are classified on the basis of their costs.

Q. 14. The property of a zeological formation, which represents its capability to allow transmission of water through it, is called :

(*a*) porosity (*b*) permeability
(*c*) field capacity (*d*) yield.

Q. 15. The water consumed in irrigation, when compared with the total water used for all purposes in our country, is about :

(*a*) 30% (*b*) 50%
(*c*) 70% (*d*) 90%.

Q. 16. 1 Tonne of wheat produced, may consume water, as much as, say :

(*a*) 100 tonnes (*b*) 500 tonnes
(*c*) 2000 tonnes (*d*) 4000 tonnes.

Q. 17. The velocity of flow of water (v) through a soil stratum of permeability K and porosity n, as per Darcy's equation, is given by :

(*a*) $v = Ki$ (*b*) $v = Ki\,A$

(*c*) $v = K\,i{\cdot}n$ (*d*) $v = Ki \cdot \frac{1}{n}$.

Q. 18. Water required in producing 1 tonne of Rice is of the order of :

(*a*) 100 tonnes (*b*) 500 tonnes

(*c*) 1000 tonnes (*d*) 4000 tonnes.

Q. 19. The zeological formation, which does not contain any amount of ground water, is an :

(*a*) aquitard (*b*) aquifer

(*c*) aquiclude (*d*) aquifuse.

Q. 20. An example of aquitard stratum in the ground profile, is a layer of :

(*a*) granite rock (*b*) clay

(*c*) sandy clay (*d*) sand stone.

Q. 21. The appropriate percentage that represents the world's water resources existing in the form of ground water, is :

(*a*) 2% (*b*) 12%

(*c*) 22% (*d*) 47%.

Q. 22. In a meandering river reach, the shallowest portions of the river can be searched out, at :

(*a*) the crossings (*b*) the convex banks

(*c*) the concave banks (*d*) none of them.

Q. 23. The 'sinuosity' of a meandering river is defined as :

(*a*) $\frac{\text{meander length}}{\text{meander width}}$ (*b*) $\frac{\text{meander width}}{\text{meander length}}$

(*c*) $\frac{\text{length of the river}}{\text{axial length of the river}}$ (*b*) $\frac{\text{axial length of the river}}{\text{length of the river}}$.

Q. 24. 'Crossings' in a meandering river, are :

(*a*) straight channel reaches, between two consecutive clockwise and anticlockwise loops

(*b*) apex points of clockwise loops

(*c*) apex points of anticlockwise loops

(*d*) both (*b*) and (*c*).

Q. 25. A river reach downstream of a newly built dam, may behave as :

(*a*) aggrading (*b*) degrading

(*c*) virgin (*d*) all of them.

Q. 26. An unconfined aquifer in ground water profile is also known, as :

(*a*) an artesian aquifer (*b*) a perched aquifer

(*c*) a water table aquifer (*d*) none of the above.

Q. 27.* The upper limit of Reynold's number, for the Darcy's law to remain valid for ground water flow, is :

(*a*) 0 (*b*) 1

(*c*) 100 (*d*) ∞.

Q. 28. The safe depression head for open wells is generally taken to be x times the critical depression head, when x is :

(*a*) $\frac{1}{6}$ (*b*) $\frac{1}{3}$

(*c*) $\frac{1}{2}$ (*d*) $\frac{3}{4}$.

Q. 29. When groundwater flows into a stream to augment its discharge, then the stream is called :

(*a*) an influent stream (*b*) an effluent stream
(*c*) a perennial stream (*d*) an ephemeral stream.

Q. 30. A bridge is proposed to be constructed across the Ganga river at a site, where the design discharge is about 20,650 cumecs. The river width is proposed to be reduced at the bridge site to 800 m by constructing guide banks and marginal bunds, on either side. The length of each guide bank in this case will be about :

(*a*) 0.8 km (*b*) 1.2 km
(*c*) 1.4 km (*d*) 2.8 km.

Q. 31.* A guide bank is to be constructed for an alluvial river, having maximum design discharge of 6400 cumecs. The thickness of the stone pitching required on the river side of this guide bank, will be about :

(*a*) 0.6 m (*b*) 0.9 m
(*c*) 1.2 m (*d*) 1.5 m.

Q. 32. A fully developed pilot cut, excavated in a river, is called :

(*a*) a cut off (*b*) a short cut
(*c*) a meander (*d*) a river channel.

Q. 33. Retrogression occurring at the downstream reach of a weir site, is :

(*a*) a continuous scouring process
(*b*) a continuous silting process
(*c*) a reversible process, in which gradual scouring occurs during the initial 4-5 years of weir construction ; which later, changes into gradual silting, and recovery towards original levels, and even more, sometimes.
(*d*) a reversible process, in which gradual silting occurs during the initial 4-5 years of weir construction ; which then changes into gradual scouring, and recovery towards original levels and even more, sometimes.

Q. 34. A 'breast wall' is usually provided above the sill of a canal head regulator at a barrage site, between :

(*a*) sill level and pond level
(*b*) pond level and HFL of river
(*c*) pond level and FSL of canal
(*d*) none of the above.

Q. 35. The alkali salt, which is most injurious to plant growth, is :

(*a*) sodium corbonate (*b*) sodium chloride
(*c*) sodium sulphate (*d*) sodium nitrate.

Q. 36. The joints in the C.C. blocks laid over the inverted filter infront of the downstream end of concrete floor of a barrage, should be filled up with :

(*a*) C.C. 1 : 2 : 4 (*b*) cement mortar 1 : 4
(*c*) bajri or coarse sand (*d*) any of the above.

Q. 37. A deflecting groyne is :

(*a*) a small repelling groyne, giving only local protection, without repelling the river current
(*b*) a small attracting groyne
(*c*) a specially designed curved head groyne
(*d*) a tree groyne.

Q. 38. The spacing between the groynes constructed to protect a dike, in the absence of model testing, may be of the order of :

(*a*) 2 to 2.5 times their length
(*b*) 8 to 10 times their length
(*c*) 20 to 30 times their length
(*d*) 50 times their length.

Q. 39. Rigid boundary canals, whose bed and banks are made with non-erodible materials, are in :

(*a*) initial regime (*b*) final regime
(*c*) true regime (*d*) permanent regime.

Q. 40. Lacey's silt theory is not applicable to channels, which are in :

(*a*) final regime (*b*) permanent regime
(*c*) true regime (*d*) none of the above.

Q. 41. A perfectly lined canal is a :

(*a*) mobile boundary canal (*b*) rigid boundary canal
(*c*) neither (*a*) nor (*b*) (*d*) either (*a*) or (*b*).

Q. 42.* The total water requirement for certain irrigated area is 300 M cum. The reservoir losses and carry over losses are 10% and 5% respectively ; and dead storage is 10% of gross storage. The gross storage capacity of reservoir should be :

(*a*) 375 M cum (*b*) 383 M cum
(*c*) 395 M cum (*d*) 400 M cum.

Q. 43. The stone apron laid infront of a hydraulic structure in the river bed, to ensure its launching into any possible scour that may occur at the site, is called :

(*a*) launching apron (*b*) rip rap
(*c*) talus (*d*) all of these.

Q. 44. A pilot cut in a meandering river should be as deep as possible, and the ratio $\frac{R}{L^2}$ for the cut should be :

(*a*) lesser for the cut than for the main course
(*b*) greater for the cut than for the main course
(*c*) equal for the cut and the main course
(*d*) none of the above.

where R is the hydraulic radius
L is the channel length.

Q. 45. Cut off ratio for an alluvial river is always :

(*a*) < 1 (*b*) 1
(*c*) > 1 (*d*) none of them.

Q. 46. When other conditions are favourable, a natural cut off along an existing side channel may develop across a sharp meandering bend in an alluvial river, at a meander ratio of the order of :

(*a*) 0.5 to 1.0 (*b*) 1.0 to 1.5
(*c*) 1.7 to 3.0 (*d*) 10 to 20.

Q. 47. The arrangements made along a canal, to help the unfortunate victims being swept away by water current, are called :

(*a*) canal crossings (*b*) cattle crossings
(*c*) canal ladders (*d*) none of them.

Q. 48. The only irrigation module, which is not an orifice type but is of weir type, is :
(*a*) Kennedy's gauge outlet (*b*) open flume outlet
(*c*) pipe outlet (*d*) adjustable proportional module.

Q. 49. The one among the following, which is not a rigid module, is :
(*a*) Gibb's module (*b*) Foote module
(*c*) Khanna's module (*d*) Kennedy's module.

Q. 50. Flood waters of rivers are sometimes diverted through canals, called :
(*a*) feeder canals (*b*) inundation canals
(*c*) perennial canals (*d*) carrier canals.

Q. 51. A cut off across a meander loop in an alluvial river leads to :
(*a*) silting of sediment upstream and scouring downstream
(*b*) scouring upstream and silting downstream
(*c*) scouring in the upstream reach only
(*d*) silting in the downstream reach only.

Q. 52. The sill level of a canal head regulator is kept higher by about 1.2 to 1.5 m, above the :
(*a*) crest level of under-sluice ways
(*b*) crest level of barrage bays
(*c*) canal bed level
(*d*) none of these.

Q. 53. The tunnel openings provided parallel to the axis of the main canal head regulator and in front of that, are called :
(*a*) head sluices (*b*) under sluices
(*c*) silt ejector (*d*) silt excluder.

Q. 54. Out of the following, the measures which help in improving the duty of irrigation water, are :
(*a*) lining of canals to reduce conveyance losses of irrigation water
(*b*) minimising water losses in irrigation fields, by improving the efficiency of cultivation methods
(*c*) lining of water courses to avoid seepage of water
(*d*) educating the farmers with the harmful effects of using excess water in irrigation
(*e*) rotation of crops in farms
(*f*) all of the above (*g*) none of the above.

Q. 55. An irrigation canal, flowing under pressure below a drainage channel, is specifically called a :
(*a*) canal aqueduct (*b*) siphon aqueduct
(*c*) super passage (*d*) canal siphon.

Q. 56. A venturiflume, which is helpful in measuring canal discharge, is also called a :
(*a*) standing wave flume (*b*) parshall flume
(*c*) non-metering flume (*d*) none of the above.

Q. 57. The drainage water intercepting the canal can be displosed of by passing the canal above the drainage, in :
(*a*) an aqueduct or a siphon aqueduct
(*b*) a super passage or a canal siphon

(*c*) a level crossing
(*d*) none of the above.

Q. 58. A level crossing constructed at the junction of an irrigation canal and a natural drainage channel consists of :
(*a*) one regulator only
(*b*) two weirs only
(*c*) one weir and two regulators
(*d*) two weirs and two regulators.

Q. 59. Which of the following is not true about a standing wave flume, used for metering purposes in a canal ?
(*a*) it involves low head loss
(*b*) its discharge depends only upon the upstream water level
(*c*) for the same discharging head, its discharge is more than that of a venturi flume
(*d*) it involves the formation of a hydraulic jump downstream of the throat
(*e*) it is seldom constructed as independent structure solely for metering purposes
(*f*) it involves low energy loss.

Q. 60. Cut offs are promoted along a river course in plains, to shorten and straighten its course. This exercise helps us to :
(*a*) dissipitate excess valley slope of upper mountainous river reach
(*b*) remedy the deficit surface slope of the river reach in plain
(*c*) both (*a*) and (*b*)
(*b*) neither (*a*) nor (*b*).

Q. 61. Out of the following, which measure helps in reducing scouring and silting along an alluvial river course :
(*a*) to dissipate excess land slope of upper reach, by constructing weirs and barrages across the river in this reach
(*b*) to overcome the deficit slope in lower plain reaches, by promoting and affecting cut offs
(*c*) by promoting soil conservation measures in the upper catchment
(*d*) by creating storage on the upper reaches, to release somewhat constant outflow in the downstream river.
(*e*) all of the above.

Q. 62. The floor of a siphon aqueduct needs to be designed for conditions, when :
(*a*) the canal and drainage are flowing full
(*b*) the canal is full and there is no drainage discharge
(*c*) the canal is empty and drainage discharge is full
(*d*) (*a*) and (*b*) both
(*e*) (*b*) and (*c*) both
(*f*) (*c*) and (*a*) both.

Q. 63.* The discharge from a spillway on a horizontal floor is 8 cumecs per metre width. If the depth before the jump is 50 cm, the depth after the jump will be ... metres. *(GATE - 92)*

Q. 64.* 'Eddy viscosity', means that it is
(*a*) a physical property of the fluid
(*b*) same as kinematic viscosity (*c*) always associated with laminar flow
(*d*) an apparent viscosity due to turbulent nature of flow. *(GATE-90)*

Q. 65. In a siphon aqueduct provided with a depressed floor, the worst uplift acting on the floor is caused when there is no drainage flow, by the
(*a*) static head due to watertable rising up to the drain bed
(*b*) seepage head due to canal running full
(*c*) both (*a*) and (*b*)
(*b*) none of the above.

Q. 66.* The following hydraulic data is available at the proposed site of a canal crossing :

Item	*Drain*	*Canal*
B.L. (m)	207.4	209.4
HFL/FSL (m)	209.3	215.6
Discharge (m^2/s)	390	350

The appropriate economical cross-drainage work at the above site will be :
(*a*) an aqueduct (*b*) a super passage
(*c*) an aqueduct syphon (*d*) a canal syphon.

Q. 67. Multiple arch dam is an example of :
(*a*) arch dams (*b*) buttress dams
(*c*) shell-arch dams (*d*) all of the above.

Q. 68. An aquifer, which is confined at its bottom but not at the top, is called a :
(*a*) semiconfined aquifer (*b*) confined aquifer
(*c*) unconfined aquifer (*d*) none of the above.

Q. 69.* The permeability of a soil sample at standard temperature of 20°C was 0.01 cm/s. The permeability at 40°C temperature, when kinematic viscosity of water at 20°C and 40°C are given to be 1.009$\mu m^2/s$ and 0.654$\mu m^2/s$, will be:
(*a*) 0.01 cm/s (*b*) 0.015 cm/s
(*c*) 0.0065 cm/s (*d*) none of these.

Q. 70. The volume of water (cum) that can be extracted from a unit volume of aquifer material (1 cum) under the force of gravity, is called :
(*a*) storage coefficient (*b*) specific coefficient
(*c*) specific yield (*d*) specific capacity.

Q. 71.* Specific capacity of a confined well under equilibrium conditions and within the working limites of drawdown :
(*a*) is constant at all drawdowns
(*b*) decreases as the drawdown is increased
(*c*) increases as the drawdown is increased
(*d*) none of the above.

Q. 72. A soil sample, having exchangeable sodium percentage = 13%, pH = 7.5, and electrical conductivity = 6000 $\mu\Omega$/cm, can be classified as :
(*a*) white alkali (*b*) alkaline soil
(*c*) saline-alkali soil (*d*) none of the above.

Q. 73. The dead storage in a reservoir is created for :
(*a*) sedimentation (*b*) firm hydropower
(*c*) emergent water supply (*d*) none of these.

Q. 74. The silt suspended in water, settles down, approximately following the :
(*a*) Blaney criddle formula (*b*) Darcy's law
(*c*) Lacey's theory (*d*) Stoke's law.

Q. 75. Sedimentation over long periods of time, is responsible for reducing the reservoir capacity, earmarked as :

(*a*) dead storage (*b*) live storage
(*c*) both (*a*) and (*b*) (*d*) none of these.

Q. 76. The most appropriate value of the 'average estimated life' of a concrete gravity or an earthen dam, is :

(*a*) 50 years (*b*) 100 years
(*c*) 150 years (*d*) none of these.

Q. 77. The axis of a gravity dam is the :

(*a*) line of the crown of the dam on the downstream side
(*b*) line of the crown of the dam on the upstream side
(*c*) centre line of the top width of the dam
(*d*) line joining mid points of the base.

Q. 78. Tension cracks in gravity dams may sometimes lead to the failure of the structure, by :

(*a*) sliding of the dam at the cracked section
(*b*) overturning about the toe
(*c*) crushing of concrete, starting from the toe
(*d*) none of the above.

Q. 79. Presence of tail water in a gravity dam :

(*a*) increases the principal stress and decreases the shear stress
(*b*) increases the principal stress and increases the shear stress
(*c*) decreases the principal stress and increases the shear stress
(*d*) decreases the principal stress and decreases the shear stress.

Q. 80. Neglecting tail water effect, the major principal stress at the toe of a gravity dam under full reservoir condition, is given by :

(*a*) p_v (*b*) $p_v \sec^2 \theta$
(*c*) $p_v \tan^2 \theta$ (*d*) $p_v \sin^2 \theta$.

where p_v = direct vertical stress at the toe
θ = angle of the downstream face with the vertical.

Q. 81. The provision of a drainage gallery in a gravity dam helps in reducing :

(*a*) hydrostatic pressure (*b*) silt pressure
(*c*) seepage pressure (*d*) none of these.

Q. 82. The uplift pressure at the face of a drainage gallery in a gravity dam, is taken as equal to :

(*a*) hydrostatic pressure at the toe
(*b*) average of hydrostatic pressure at the toe and heel
(*c*) two third of the hydrostatic pressure at the toe plus one third of the hydrostatic pressure at the heel
(*d*) none of the above.

Q. 83. The pore pressure distribution under steady condition in an earthen dam, is:

(*a*) determined by considering linear variation from upstream to downstream water levels
(*b*) determined from the flownet
(*c*) the same as the upstream water level
(*d*) none of the above.

Q. 84. A rock toe and a horizontal filter is provided on the downstream base of an earthen dam in order to :
(*a*) prevent piping action in the dam body
(*b*) prevent piping action in the dam foundation
(*c*) to reduce the seepage quantity by blocking its flow
(*d*) to collect and drain out the seepage flow.

Q. 85. The one among the following, which helps to meet the downstream river water requirements in a dam reservoir project, is :
(*a*) dam spillway (*b*) dam galleries
(*c*) dam sluices (*d*) none of these.

Q. 86. The crest of a dam spillway is provided at R.L. equal to that of its :
(*a*) maximum reservoir level
(*b*) dead storage level
(*c*) maximum conservation level
(*d*) none of the above.

Q. 87. The seepage discharge in an earthen dam, as collected by a horizontal blanket or a rock toe, can be computed by :
(*a*) plotting the flow net below the dam body
(*b*) plotting the flow net through the dam body
(*c*) either (*a*) or (*b*)
(*d*) none of the above.

Q. 88. The hydraulic jump for energy dissipation is used in :
(*a*) shaft spillway
(*b*) syphon spillway
(*c*) side channel spillway
(*d*) none of them.

Q. 89. The formation of a hydraulic jump at the foot of a spillway is one of the common methods of energy dissipation services, because :
(*a*) it destroys more than 90% of the total energy by the turbulence produced in the jump
(*b*) it reduces kinetic energy by increasing the depth of flow
(*c*) its action is not understood. *(U.P.S.C. Engg. Services, 1977)*

Q. 90. The process by virtue of which water moves from regions of higher concentration to lower concentration, the two regions being separated by a selectively permeable membrane, is called :
(*a*) diffusion (*b*) osmosis
(*c*) transpiration (*d*) reverse osmosis.

Q. 91. The productivity of a soil will be maximum, when its pH value is of the order of :
(*a*) below 6 (*b*) 6.5—7.5
(*c*) 7.5—8.0 (*d*) 8.5— 11.0.

Q. 92.* The efficiency of sprinklar irrigation is the highest in :
(*a*) arid regions (*b*) semi arid regions
(*c*) humid regions (*d*) none of them.

Q. 93. The uniformity coefficient will be higher in case of :
(*a*) drip irrigation (*b*) sprinkler irrigation
(*c*) equal in both cases
(*d*) sometimes (*a*) and sometimes (*b*), depending on soil type.

Q. 94. The structure among the following, which is not used in a shaft spillway, is:
(*a*) a tunnel (*b*) a bridge
(*c*) radial gates (*d*) radial piers.

Q. 95. 'Morning glory' is the :
(*a*) special flared inlet of the shaft spillway of a dam of very small height
(*b*) special flared inlet of the shaft spillway of a large dam project
(*c*) horizontal tunnel constructed in a shaft spillway across the body of a gravity dam, to carry the surplus reservoir water to the downstream river
(*d*) horizontal tunnel constructed in the shaft spillway of an earthen dam through its foundation, to carry the surplus reservoir water to the downstream river.

Q. 96. A ski-jump bucket is also known as :
(*a*) flip bucket (*b*) solid roller bucket
(*c*) slotted roller bucket (*d*) none of these.

Q. 97. The most commonly used vertical lift gates in modern days is :
(*a*) sliding gates (*b*) free-roller gates
(*c*) stoney gates (*d*) fixed wheel gates.

Q. 98. The percentage of energy dissipation in a hydraulic jump :
(*a*) increases with the increase in the Froude number (of the incoming flow)
(*b*) decreases with the increase in the Froude number
(*c*) remains unaffected with the increase in the Froude number
(*d*) increases with the increase in Froude number up to a limit, and then decreases with further increase in Froude number.

Q. 99. In case of dam spillways, the approaching Froude number F_1, usually lies in the range of :
(*a*) < 2.5 (*b*) 2.5—4.5
(*c*) 4.5—0.9, or sometimes even more
(*b*) none of them.

Q. 100. In case of weirs and barrages, the approaching Froude number F_1, usually lies in the range of :
(*a*) < 2.5 (*b*) 2.5—4.5
(*c*) 4.5—9.0 (*d*) more than 9.0.

ANSWERS FOR THE TEST PAPER

1. (*c*)	2. (*b*)	3. (*c*)	4. (*c*)	5. (*c*)
6. (*a*)	7. (*d*)	8. (*c*)	9. (*d*)	10. (*d*)
11. (*d*)	12. (*b*)	13. (*c*)	14. (*b*)	15. (*d*)
16. (*c*)	17. (*d*)	18. (*d*)	19. (*d*)	20. (*c*)
21. (*c*)	22. (*b*)	23. (*c*)	24. (*a*)	25. (*b*)
26. (*c*)	27. (*b*)	28. (*b*)	29. (*b*)	30. (*c*)
31. (*c*)	32. (*a*)	33. (*c*)	34. (*b*)	35. (*a*)
36. (*c*)	37. (*a*)	38. (*a*)	39. (*d*)	40. (*b*)
41. (*b*)	42. (*c*)	43. (*d*)	44. (*b*)	45. (*c*)
46. (*c*)	47. (*c*)	48. (*b*)	49. (*d*)	50. (*b*)
51. (*b*)	52. (*a*)	53. (*d*)	54. (*f*)	55. (*d*)
56. (*b*)	57. (*a*)	58. (*c*)	59. (*a*)	60 (*b*)

61. (*e*)	62. (*e*)	63. 4.86 m	64. (*d*)	65. (*c*)
66. (*c*)	67. (*b*)	68. (*c*)	69. (*b*)	70. (*c*)
71. (*b*)	72. (*a*)	73. (*a*)	74. (*d*)	75. (*c*)
76. (*c*)	77. (*b*)	78. (*c*)	79. (*d*)	80. (*b*)
81. (*c*)	82. (*c*)	83. (*b*)	84. (*d*)	85. (*c*)
86. (*c*)	87. (*b*)	88. (*c*)	89. (*b*)	90. (*b*)
91. (*b*)	92. (*c*)	93. (*a*)	94. (*c*)	95. (*b*)
96. (*a*)	97. (*d*)	98. (*a*)	99. (*c*)	100. (*b*).

HINTS FOR SOLVING STARRED QUESTIONS OF THE TEST PAPER

Q. 5. From Fig. 30.1, it can be seen that the original berm AB attains $A'B'$, when channel attains $\frac{1}{2}H:1V$ regime side slopes. From geometry of this Fig., $A'B'$ is found to be $1.2D$.

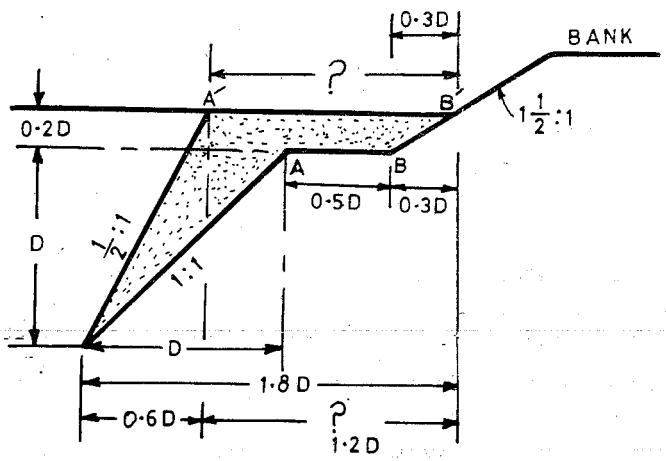


Fig. 30.1.

Q. 6. From eqn. (1.2)

$$A_{max} = \frac{Q}{f}$$

where $Q = 0.05$ cumec

$= 0.05 \times 60 \times 60\ m^3/h$

$= 180\ m^3/h$

$f = 5$ cm/h $= 0.05$ m/hr

$$= \frac{180}{0.05}\ m^2 = 3600\ m^2$$

$= 0.36$ hectare. **Ans.**

Q. 8.

$$\tan\theta = \frac{1}{1.25}$$

$$\theta = 0.675$$

From eqn. (5.8)

$$A = (B + y\theta + y \cot \theta)$$
$$= 2\,(12 + 2 \times 0.675 + 2 \times 1.25) = 31.7\ \text{m}^2$$

From eqn. (5.9)

$$P = B + 2y\theta + 2y \cot \theta$$
$$= 12 + 2 \times 2 \times 0.675 + 2 \times 2 \times 1.25 = 19.7\ \text{m}$$
$$R = \frac{A}{P} = \frac{31.2}{19.7} = \mathbf{1.61\ m.} \quad \textbf{Ans.}$$

Q. 11. $M \approx 150\sqrt{Q_{dom}}$

$$= 150 \times \sqrt{5000 \times \frac{9}{16}}\ \text{m} \approx 8\ \text{km}$$

Spacing provided = 2 km

Meander belt cut off

$$= 8 - 2 = 6\ \text{km}$$
$$= \frac{6}{8} \times 100\% = 75\% \quad \textbf{Ans.}$$

Q. 12. $M_B = 3400\ \text{m} \approx 17\ \text{W}$

$\therefore$ $W = 200\ \text{m.}$ **Ans.**

Q. 27. $\because$ The flow becomes turbulent when R_e exceeds 1, and Darcy's law is valid only in laminar range.

Q. 31. From Eqn. (8.4) : $t = 0.06\,(6400)^{1/2}$

$$= 1.11 \approx 1.2\ \text{m.} \quad \textbf{Ans.}$$

Q. 42. Carryover (conveyance) loss is 5% (of head discharge)

$\because$ Water drawn from reservoir

$$= \frac{300}{0.95}\ \text{M cum.}$$

Reservoir loss and dead storage

$$= 20\%$$

$\because$ Gross reservoir capacity

$$= \frac{\text{water drawn}}{0.8}$$
$$= \frac{300}{0.95} \times \frac{1}{0.8} = 394.7\ \text{M cum}$$
$$\approx 395\ \text{M cum} \quad \textbf{Ans.}$$

Q. 63. From eqn. (10.6) :

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$$

where $q = 8$ cumecs/m

$y_1 = 0.5$ m

$\therefore$
$$y_2 = -\frac{0.5}{2} + \sqrt{\frac{0.25}{4} + \frac{2 \times 64}{9.81 \times 0.5}}$$
$$= 4.86\ \text{m.} \quad \textbf{Ans.}$$

Q. 64. See article 4.8.

Q. 66. Refer Example 14.5 (*b*).

Q. 69.
$$K \propto \frac{1}{\nu}$$

$$\therefore \quad \frac{K_{40°}}{K_{20°}} = \frac{\nu_{20°}}{\nu_{40°}} = \frac{1.009}{0.654}$$

$$\therefore \quad K_{40°} = \frac{1.009}{0.654} \times 0.01 \text{ cm/s} = 0.015 \text{ cm/s}. \quad \textbf{Ans.}$$

Q. 71. From eqn. (16.28) :
Specific capacity
$$= \frac{1}{C_1 + C_2 Q}.$$
This eqn. shows that specific capacity decreases as Q increases, and Q increases only when drawdown is increased by heavier pumping, and hence, specific capacity decreases as drawdown is increased. **Ans.**

Q. 92. Efficiency of sprinkler irrigation is the highest in humid or cool climates.

31

Important Information to Students on UPSC Competitions and Conventional Questions of the Past Years

The Union Public Service Commission, Dholpur House, Shahjahan Road, New Delhi-110011, conducts every year two competitions, in which, you as civil engineering graduates (even with exam. results awaited) can participate for securing a direct class I Govt. appointment.

The first competition is named as **Combined Engineering Services (CES) exam,** and the other competition is known as **Civil Services Exam.**, popularly called *IAS competition.*

Press notifications for obtaining and submitting application forms are released by Secretary UPSC, through news papers including "Employment News" sometimes in Feb. every year. The exams are usually held in August.

On the basis of the first competition, you are likely to be appointed as Asstt. Executive Engineer or Asstt. Director or on an equivalent post in civil engineering side of the various Government Departments like Railways, CPWD, P&T, Central Water Commission, etc. whereas, on the basis of the second competition, you are liable to become an IFS, IAS, IPS, or an officer on allied services, such as Asstt. Custom Collector or Asstt. Excise Collector, etc.

Most of the engineering degree students aspire to clear any one of these two competitions, for an excellent career full of promotional avenues, just after doing BE degree. The syllabus for both these competitions, now a days, provide full opportunities to Civil Engineering degree holders, to make through these competitions.

We will here discuss the syllabus and examination schemes as may be of use to you as civil engineering graduates, for both of these competitions.

COMBINED ENGINEERING SERVICES EXAM. (Civil Engg.)

In this competition, there will be 5 papers in all, 3 being of objective type, and 2 of conventional type. Each paper carries 200 marks, with a total of 1000 marks. 200 marks are reserved for Personal Interview, making a grand total of 1200 marks, as below:

Section I — Objective Type Papers		
1. *General Ability Test*	2 hr.	200 marks
2. *Civil Engineering Paper I*	2 hr.	200 marks
3. *Civil Engineering Paper II*	2 hr.	200 marks
Section II — Conventional Papers*		
4. *Civil Engineering Paper I*	3 hr.	200 marks
5. *Civil Engineering Paper II*	3 hr.	200 marks
	Total	1000 marks
*Personality Test cum Interview** (only for those candidates who qualify on the basis of the written examination)		200 marks
	Grand Total Marks	1200

The syllabus for these papers (revised wef year 2000) is given below :

Syllabus for General Ability Test (Objective Type)

Part A — *General English.* The question paper in General English will be designed to test the Candidate's understanding of English and workman like use of words.

Part B — *General Studies.* The paper in General studies will include knowledge of current events and of such matters as of every day observation and experience in their scientific aspects as may be expected of an educated person. The paper will include questions on History of India and Geography of a nature which candidates should be able to answer without special study.

Syllabus for Civil Engineering Paper I (Revised wef the year 2000)

(Same as for Objective as well as Conventional Papers)

1. **Building Materials**

Timber. Different types and species of structural timber, density-moisture relationship, strength in different directions, defects, influence of defects on permissible stress, preservation, dry and wet rots, codal provisions for design, Plywood.

Bricks. Types, Indian Standard classification, absorption, saturation factor, strength in masonry, influence of mortar strength on masonry strength.

Cement. Compounds of, different types, setting times, strength.

Cement Mortar. Ingredients, proportions, water demand, mortars for plastering and masonry.

Concrete. Importance of W/C Ratio, Strength, ingredients including admixtures, workability, testing for strength, elasticity, non-destructive testing, mix design methods.

2. **Solid Mechanics.** Elastic constants, stress, plane stress, Mohr's circle of stress, strains, plane strain, Mohr's circle of strain, combined stress; Elastic theories of failure; Simple bending, Shear, Torsion of circular and rectangular sections and simple members.

3. **Structural Analysis.** Analysis of determinate structures - different methods including graphical methods.

Analysis of indeterminate skeletal frames - moment distribution, slope-deflection, stiffness and force methods, energy methods, Muller-Breslau principle and application.

Plastic analysis of indeterminate beams and simple frames-shape factors.

* In the personality test special attention will be paid to assessing the candidates capacity for leadership, initiative and intellectual qualities, mental & physical energy, powers of practical application and integrity of character.

4. **Design of Steel Structures.** Principles of working stress method. Design of connections, simple members; Built-up sections and frames, Design of Industrial roofs. Principles of ultimate load design. Design of simple members and frames.

5. **Design of Concrete and Masonry Structures.** Limit state design for bending, shear, axial compression and combined forces. Codal provisions for slabs, beams, walls and footings. Working stress method of design of R.C. members.

Principles of prestressed concrete design, materials, methods of prestressing, losses. Design of simple members and determinate structures. Introductions to prestressing of indeterminate structures.

Design of brick masonry as per I.S. Codes.

6. **Construction Practice, Planning and Management.**

Concreting Equipment : Weight Batcher, Mixer, vibrator, batching plant, concrete pump. Cranes, hoists, lifting equipment.

Earthwork Equipment : Power shovel, hoe, dozer, dumper, trailers and tractor, rollers, sheep foot rollers, pumps.

Construction, Planning and Management : Bar chart, linked bar chart, work-break down structures, Activity - on - arrow diagrams. Critical path, probabilistic activity durations ; Event-based net-works.

PERT network : Time-cost study, crashing; Resource allocation.

Syllabus for Civil Engg. Paper II (Revised wef year 2000)

1. (*a*) **Fluid Mechanics, Open Channel Flow, Pipe Flow.** Fluid Properties, pressure, Thrust, Buoyancy; Flow Kinematics; Integration of flow equations; Flow measurement; Relative motion; Moment of momentum; Viscosity, Boundary layer and Control, Drag, Lift; Dimensional Analysis, Modelling; Cavitation; Flow oscillations; Momentum and Energy principles in Open channel flow, Flow controls, Hydraulic jump, Flow sections and properties; Normal flow, Gradually varied flow; Surges; Flow development and losses in pipe flows, Measurements; Siphons; Surges and Water hammer; Delivery of Power Pipe networks.

(*b*) **Hydraulic Machines and Hydropower.** Centrifugal pumps, types, performance parameters, scaling, pumps in parallel; Reciprocating pumps, air vessels, performance parameters; Hydraulic ram; Hydraulic turbines, types, performance parameters, Controls, choice; Power house, classification and layout, storage, pondage, control of supply.

(2) (*a*) **Hydrology.** Hydrological cycle, precipitation and related data analyses, PMP, Unit and synthetic hydrographs; Evaporation and transpiration; Floods and their management, PMF; Streams and their gauging; River morphology; Routing of floods; Capacity of Reservoirs.

(*b*) **Water Resources Engineering.** Water resources of the globe : Multipurpose uses of Water : Soil-Plant-Water relationships, Irrigation systems, Water demand assessment; Storages and their yields, Ground water yield and well hydraulics; Water logging, drainage design; Irrigation revenue; Design of rigid boundary canals, Lacey's and Tractive force concepts in canal design, Lining of canals; Sediment transport in canals; Non-Overflow and overflow sections of gravity dams and their design, Energy dis-

sipators and tail water rating; Design of headworks, distribution works, falls, cross-drainage works, outlets; River training.

(3) (*a*) **Water Supply Engineering.** Sources of supply, yields, Design of intakes and conductors; Estimation of demand; Water quality standards; Control of Water-borne diseases; Primary and secondary treatment, detailing and maintenance of treatment units; Conveyance and distribution systems of treated water, leakages and control; Rural water supply; Institutional and industrial water supply.

(*b*) **Waste Water Engineering.** Urban rain water disposal; Systems of sewage collection and disposal; Design of sewers and sewerage systems; Pumping; Characteristics of sewage and its treatment, Disposal of products of sewage treatment, Stream flow rejuvenation, Institutional and industrial sewage management; Plumbing Systems; Rural and semi-urban sanitation.

(*c*) **Solid Waste Management.** Sources, classification, collection and disposal; Design and Management of landfills.

(*d*) **Air and Noise Pollution and Ecology.** Sources and effects of air pollution, monitoring of air pollution; Noise pollution and standards; Ecological chain and balance, Environmental assessment.

(4) (*a*) **Soil Mechanics.** Properties of soil, classification and interrelationship; Compaction behaviour, methods of compaction and their choice; Permeability and seepage, flow nets, Inverted filters; Compressibility and consolidation; Shearing resistance, stresses and failure; soil testing in laboratory and in-situ; stress path and applications. Earth pressure theories, stress distribution in soil; soil exploration, samplers, load tests, penetration tests.

(*b*) **Foundation Engineering.** Types of foundations, Selection criteria, bearing capacity, settlement, laboratory and field tests; Types of piles and their design and layout, Foundations on expansive soils, swelling and its prevention, foundation on swelling soils.

5. (*a*) **Surveying.** Classification of surveys, scales, accuracy; Measurement of distances - direct and indirect methods; optical and electronic devices; Measurement of directions, prismatic compass, local attraction; Theodolites - types; Measurement of elevations - Spirit and trigonometric levelling; Relief representation; Contours; Digital elevation modelling concept; Establishment of control by triangulations and traversing - measurements and adjustment of observations, computation of coordinates; Field astronomy, Concept of global positioning system; Map preparation by plane tabling and by photogrammetry; Remote sensing concepts, map substitutes.

(*b*) **Transportation Engineering.** Planning of highway systems, alignment and geometric design, horizontal and vertical curves, grade separation; Materials and construction methods for different surfaces and maintenance. Principles of pavement design; Drainage.

Traffic surveys, Intersections, signalling : Mass transit systems, accessibility, networking.

Tunnelling, alignment, methods of construction, disposal of muck, drainage, lighting and ventilation, traffic control, emergency management.

Planning of railway systems, terminology and designs, relating to gauge, track, controls, transits, rolling stock, tractive power and track modernisation; Maintenance; Appurtenant works; Containerisation.

Harbours - layouts, shipping lanes, anchoring, location identification; Littoral transport with erosion and deposition; sounding methods; Dry and Wet docks, components and operational Tidal data and analyses.

Airports - layout and orientation; Runway and taxiway design and drainage management; Zoning laws; Visual aids and air traffic control; Helipads, hangers, service equipment.

Note 1. Calculators are not allowed in objective Type papers but are allowed in conventional type papers.

Note 2. In Question papers, wherever required, SI units will be used.

A perusal of this syllabus shows that **Paper I** covers the entire Structural Engineering field, including *Building construction, Applied mechanics, Strength of materials, Theory and design of Structures* including *R.C.C., Steel, Masonry* and *Timber structures.*

Similarly, **Paper II** covers *Fluid mechanics, Irrigation and Hydrology, Water Supply and Sewage, Soil mechanics, Surveying, and Transportation.*

Both the objective papers; *i.e. Paper I (objective)* as well as *Paper II (objective)* usually contain 120 questions each, to be solved in 120 minutes. All the subjects covered in these papers usually get almost equal weightage. The objective questions from Irrigation side (*i.e.* water resources side for the years 1993 to 2002 have been given in the next chapter.

Similarly, both the conventional papers; *i.e. Paper I (conventional)* as well as *paper II (consventional*) usually respectively contain 8 questions, out of which 5 questions are to be solved. This paper-II usually carries 2 questions from Irrigation and Hydrology side, which are fully covered through this book. The other 2 to 3 questions are covered from author's other books on "Water Supply Engineering", "Sewage Disposal and Air Pollution Engineering" and "Soil-Mechanics and Foundation Engineering", thus enabling you to fetch near about cent per cent marks in this paper, thus, opening your way for selection in this competition for an excellent professional career.

You, *my dear students*, would further feel enlightened, when we inform you that in the past years, many numerical questions on water resources engineering side, have been directly set from the solved examples of this book, besides setting almost all the theoretical questions from this book.

Say for example, *in the year 1980, only two numerical questions of 80 marks were set, and both from this book in toto, without any change even in the numerical data values. In the year 1978, only one question appeared from irrigation, and that too from this book. In the year 1983, $2\frac{1}{2}$ questions were set from Irrigation, and all these were from this book* (with very slight modifications in the numerical data values), *similarly, one numerical question on 'Economics of Lining' appeared in 1986, with exactly the same data as solved in this book. In 1989, one full question on determining gross irrigation requirement for wheat crop was set from this book (with slight modifications in rainfall values), besides a question on S-hydrograph, which was also fully covered by this book. In 1993 as well as in the year 2000, all the three numerical questions on irrigation side were set from the solved examples of this book.*

In other years also, questions which appeared in this UPSC paper, were fully covered by this book.

In order to enable you to check these facts, and for your facility and guidance, given below are the questions that appeared during the last 23 years since the year 1978, from Irrigation and Hydrology side in this UPSC competition.

Questions of the Past Years Relating to Water Resources Engg. from the Engg. Services Competitions (Civil Engineering Paper II)

Year 1978 *(Engineering Services)*

Q. 5. Wheat is to be grown in a field having field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity at 80 cm depth of the soil, if

the dry unit weight of the soil is 1.5 gm/cc, if irrigation water is to be supplied when the average soil moisture falls to 18% , find the water depth required to be supplied to the field if the field application efficiency is 80%. What is the amount of water needed at the canal outlet if the water lost in the water courses and the field channel is 15% of the outlet discharge ? (40*)

[**Note.** Pl. refer solved Example 2.14. **This question was set from this book**].

Year 1979 *(Engineering Services)*

Q. 2. (*a*) Define critical tractive stress and state the main factors on which it depends. A wide unlined channel carrying silt free water has a depth of 2.0 m. The maximum tractive stress permissible on the bed to prevent scour is 0.20 kg/m^2. What is the maximum slope that can be given to the channel. (16)

[**Hint.** $\tau_0 = \gamma RS,$

$\therefore \quad \tau_{0\,(max)} = \gamma R.S_{max}$

$= 1000 \times 2.0 \times S_{max} = 0.20 \text{ kg/cm}^2$

$\therefore \quad S_{max} = 1 \text{ in } 10{,}000.$ **Ans.**]

(b) Water discharging from an overfall spillway flows to a jump type basin. The discharge intensity is 30 $m^3/s/m$ and the water depth is 1.5 m. Find the conjugate depth for the jump to form. Give the sketch of the basin showing approximate dimensions and basin appurtenances considered desirable. (17)

[**Hint.** Conjugate depth

$$= y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}$$

$$= -\frac{1.5}{2} + \sqrt{\frac{1.5^2}{4} + \frac{2 \times 30^2}{9.81 \times 1.5}}$$

$$= -0.75 + \sqrt{0.5625 + 122.324}$$

$$= -0.75 + 11.08$$

$$= 10.33 \text{ m}$$

Length of jump basin

$$= 5\,(y_2 - y_1) = 5\,[10.33 - 1.5] = 44.15 \text{ m}.$$ **Ans.**]

Or

The ordinates of a 3 hour unit hydrograph are as given :

Time, hours	0	3	6	9	12	15	18	21	24	27	30
Ordinates, m^3/s	0	10	25	20	16	12	9	7	5	3	0

Find the ordinates of 6 hour hydrograph for the same basin :

[**Note.** Please refer Example 7.33]

Year 1980 *(Engineering Services)*

In 1980, five questions, each of 40 marks, were to be attempted. There were two full questions on "Irrigation and Hydrology", which were set from this very book, as given below :

* Marks allotted for the question, out of maximum 200 marks of Civil Engg. Paper II.

Q. 3. *The rainfall runoff relation for the 655.2 sq. km. of drainage basin under dry soil conditions is given below :*

Rainfall (cm)	*Runoff (cm)*
1.5	*0.0*
5.0	*2.0*
7.5	*4.0*
10.0	*6.5*
12.5	*8.75*

A storm of 2 hr effective duration occurred over this basin at a time when the soil was dry. The resulting hydrograph was as follows :

Time (hr)	*Flow (cumecs)*
0	*0*
3	*300*
6	*500*
9	*400*
12	*250*
15	*75*
18	*0*

Determine the number of mm of runoff from this basin from this storm, assume zero base flow. Approximately how much rain was there in this storm ? What was the average coefficient of runoff during this storm ? (40)

[**Note.** Pl. refer solved Example 7.40. **This question was set from this book.** The value of C.A. has since been revised.]

Q. 6. (*a*) A 1.5 m Sarda type fall is provided at a canal with a discharge of 12 cumecs, with the following data :

Bed level upstream = *103.0 m*
Side slope of channel = *1 : 1*
Bed level downstream = *101.5 m*
Full supply level upstream = *104.5 m*
Bed width U/S & D/S = *1.0 m*
Soil = *good loam*
Bligh's coefficient = *6*

Calculate the following elements of the fall :

(i) Crest R.L.
(ii) Length of the crest and the thickness at the base.
(iii) Depth of the cistern. (40)

Or

(*b*) Calculate the elements of the fall required on Q. 6 (*a*) using as much of the data as you require. Assume data not given.

[**Note.** Pl. refer solved Example 12.3. **This question was set from this book**].

Year 1981 *(Engineering Services)*

Q. 6. (*a*) Define specific energy and critical depth. (8)

(*b*) Draw the specific energy diagram and label its various parts with brief explanation. The critical depth may also be shown. Discharge remains constant. (12)

(*c*) A trapezoidal channel has a bottom width of 6.0 m and side slopes of 1 : 1. The depth of flow is 1.5 m at a discharge of 15 m^3/sec. Determine the specific energy. (12)

(*d*) If the critical depth is 0.9 m, discuss the type of flow very briefly, corresponding to the critical depth. (8)

Q. 8. Using Rational method, rainfall-intensity duration curves (Diagram I) and the data given on diagram II, compute the diameter of the outfall sewer. The length of lines, drainage areas, and inlet times are marked on diagram II itself.

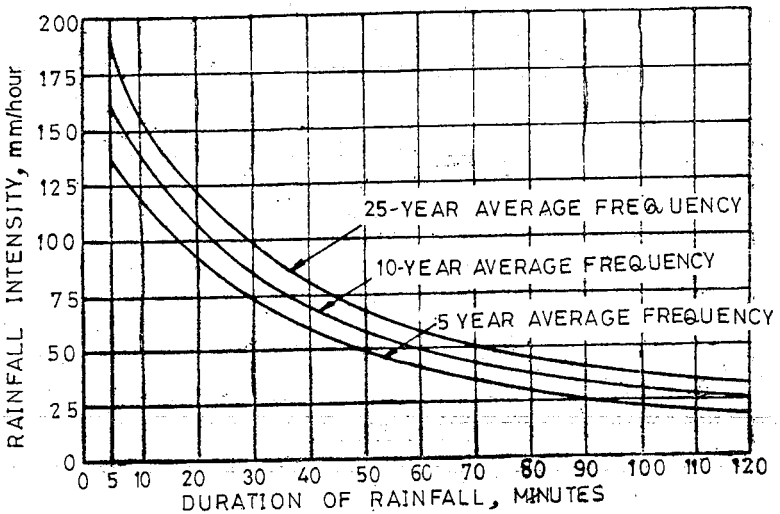


Diagram I (Fig. 31.1)

Assume :

(i) Runoff coefficient for the entire area = 0.30.

(ii) Velocity of flow in sewers flowing full = 0.75 m/sec.

(iii) 5 year average frequency curve may be used (Diagram II).

(iv) Table 1 gives the hydraulic elements for circular pipes flowing full.

Table 1. Hydraulic Elements for a Circular Pipe Flowing Full

Quantity of flow, l/s	*Dia. of pipe, mm*	*Slope of pipe, m/m*	*Velocity m/sec*
400	*450*	*0.025*	*2.7*
600	*525*	*0.020*	*2.8*
690	*1050*	*0.00055*	*0.75*
1500	*1350*	*0.001*	*1.55*
2000	*1450*	*0.001*	*1.20*

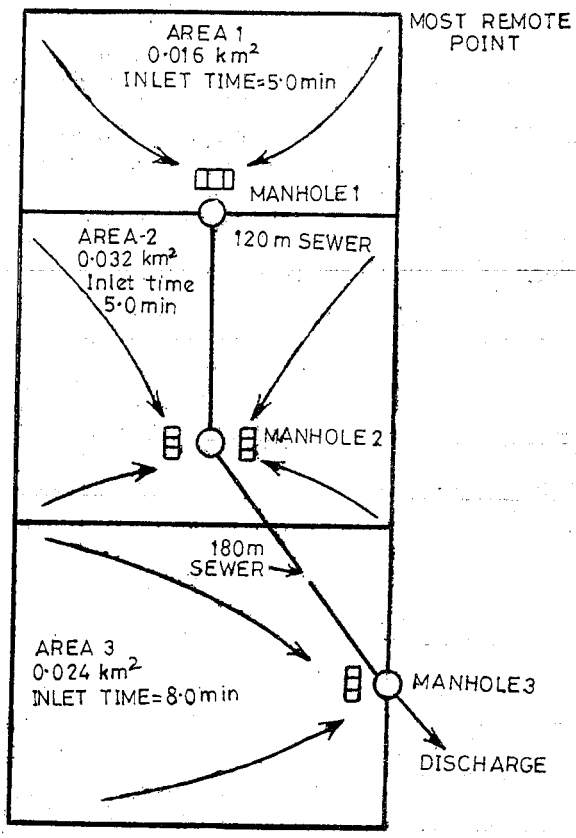


Diagram II (Fig. 31.2)

Hint. This question has been solved on Pages 790 to 791 in author's another book titled "Sewage Disposal and Air Pollution Engg." (11th Edition – 1998)

Year 1982 *(Engineering Services)*

Q. 2. *(a) Explain briefly the concept of a unit hydrograph.* (5)

(b) What is a S-curve ? What is its use ? (5)

(c) On a catchment area of 20 hectares, rainfalls of 7.5 cm, 2.0 cm and 5.0 cm occurred on three consecutive days. The average ϕ_{index} was 2.5 cm/day. The distribution graph percentages of surface runoff which extended over six days for every rainfall of such magnitudes are 5, 15, 40, 25, 10 and 5. Determine the ordinates of the discharge hydrograph and determine the peak discharge. Neglect base flow. (30)

[**Note.** Please refer Example 7.47]

Q. 3. *(a) List the various forces to be considered in the stability analysis of a gravity dam. Indicate the expressions to determine their value.* (15)

(b) Investigate the stability against overturning of a solid gravity dam at the base section. The dam is in the shape of a rightangled triangle with the upstream face vertical.

Ht. of dam = *120 m*
Slope of d/s face = *0.75 H : 1 V*
Ht. of water = *120 m*
Free-board = *0*
Ht. of tail water = *0*
Unit wt. of concrete = *2400 kg/m^3*
Uplift intensity factor = *0.5*
Allowable coeff. of friction = *0.75*

Neglect silt pressure, earthquake forces, waves forces and its pressure. Assume any suitable data as required. (25)

Year 1983 *(Engineering Services)*

Q. 1. *(b) Design a concrete lined channel to carry a discharge of 500 cumecs at a slope of 1 in 4000. The side slopes of the channel may be taken as 1 : 1. The Manning's roughness coefficient for the lining is 0.014. Assume permissible velocity in the section as 2.5 m/sec.* (20)

[**Hint.** Follow Solved Example 5.4]

Q. 2. *(a) Describe briefly how the profile of ogee-shaped spillway section is designed.* (12)

(b) The tail water rating curve for the channel below an overflow spillway is below the "jump height curve" for all expected discharges. For this case, explain with the help of neat sketches the energy dissipation arrangements which can be provided below the spillway. (16)

(c) What are the functions and design considerations of canal head regulators? (12)

Q. 4. *(a) A flood of a certain magnitude has a return period of 25 years. What is its probability of exceedence ? What is the probability that this flood may occur in the next 12 years ?* (10)

[**Note.** Please refer Example 7.52.]

(b) In a typical 4 hour storm producing 100 mm of runoff from a basin, the stream flows observed are as follows :

Time in hr.	*0*	*2*	*4*	*6*	*8*	*12*	*16*	*20*
Flow in cumecs	*0*	*2.44*	*8.10*	*13.50*	*11.34*	*6.75*	*2.70*	*0.0*

Estimate the peak flow in the stream and the time of its occurrence in a flood created by an 8 hr storm which produces 50 mm of runoff during the first four hours and 75 mm of runoff during the second four hours. Assume base flow to be negligible. (20)

[**Note.** Please refer solved Example 7.32.**This question was set from this book by just doubling the data values**].

(c) Explain how the average depth of precipitation over an area, due to a storm is computed from isohyetal map of the storm. (10)

Year 1984 *(Engineering Services)*

Q. 2. *(a) Mention the various types of canal falls commonly used indicating distinctive features, and describe briefly design principles of any one popular type.* (20)

(b) A weir has a solid horizontal floor length of 50 m with two lines of cutoff 8 m depth, below the river bed, at its two ends. The floor thickness is 1 m at the upstream end and 2 m at the downstream end, with its upper level being in flush with the river bed. For an effective head of 5 m over the weir, calculate the uplift pressure at the two inside corner points junction of bottom of floor with the cutoff), and also exit gradient. (20)

Q. 3. *(a) Describe briefly Kennedy's theory for the design of regime channels.* (10)

(b) Design a regime channel to carry a discharge of 100 cumecs in a soil having silt factor of 1.1, using Lacey's theory. Assume side slopes of 0.5 horizontal to 1.0 vertical. (18)

[**Note.** Please follow solved Example 4.9].

Q. 4. *(a) What are the basic propositions of unit hydrograph theory as propounded by L.K. Sherman ?* (10)

(b) The mass curve of precipitation resulted from the storm of 14th August 1983 gave the following values :

Hour	*Accumulated depth at the end of periods in mm*
22.00	*0.0 (Beginning of storm)*
22.05	*10.2*
22.10	*20.8*
22.15	*33.0*
22.20	*47.2*
22.25	*55.8*
22.30	*64.0*
22.35	*71.6*
22.40	*78.8*
22.45	*85.4*
22.50	*91.4 (End of storm)*

For the above storm, construct hydrograph, and draw maximum intensity duration curve. (20)

[**Note.** Please refer Example 7.11]

(c) Describe the function and design consideration of undersluices provided in diversion head works. (10)

Year 1985 *(Engineering Services)*

Q. 2 *(a) Describe briefly the main functions of a distributary 'Head Regulator' and 'Cross Regulator'. Mention the salient steps for their design.* (20)

(b) Design an irrigation channel by Kennedy's method to carry 50 cubic metres per second of discharge with base width to depth ratio as 2.5. The critical velocity ratio is

1.1. Assume Kutter's rugosity coefficient as 0.025 and side slopes of the channel as 0.5 horizontal to 1.0 vertical. (20)

[**Note.** Please follow solved Example 4.7].

Q. 3. *(a) How do you determine the storage capacity of a reservoir with the help of mass curve of runoff, if a constant or a variable demand is known ?* (12)

(b) Explain briefly the 'Infiltration Indices'. (8)

(c) In estimating the peak discharge of a river at X, the catchment area was divided into four parts A, B, C and D. The time of concentration and the area for different parts are as follows :

Part	*Time of concentration*	*Area in square metres*
A	*One hour*	600×10^4
B	*Two hours*	750×10^4
C	*Three hours*	1000×10^4
D	*Four hours*	1200×10^4

Records for a rain storm lasting for four hours as observed, and the runoff factor during different hours are as follows :

Time in hr.		*Rainfall (in mm)*	*Run-off factor*
From	*To*		
0	*1*	*25.0*	*0.50*
1	*2*	*50.0*	*0.70*
2	*3*	*50.0*	*0.80*
3	*4*	*23.5*	*0.85*

Calculate the maximum flow to be expected at X in cumecs, assuming a constant base flow of 42.5 cumecs. (20)

[**Note.** Please refer Example 7.31].

Q. 4. *(a) Explain briefly the functions and requirements of a good module. How are modules classified ? Mention common examples of each type.* (18)

(b) What are the various assumptions made in two dimensional design of gravity dams ? (12)

(c) Describe briefly uplift Pressure on Syphon Flooring in cross-drainage works. (10)

Year 1986 *(Engineering Services)*

Q. 3. *(a) Explain 'Retrogression of Downstream Levels' consequent to weir construction.* (8)

(b) Describe briefly causes of failure of earthen dams. (12)

(c) An unlined canal giving a seepage loss of 4.0 cumecs per million sq metres of wetted area is proposed to be lined with 12 cm thick cement concrete lining, which costs Rs. 20.0 per sq. m. Workout the economics of lining and show if the scheme is justified on the basis of the following data:

Annual revenue from crops = *Rs. 4,00,000 per cumec of water.*
Discharge of the channel = *50 square metre*
Wetted perimeter of the channel = *22.4 m*
Wetted perimeter of the lining = *22.0 m*
Annual maintenance cost of unlined channel = *12 paise per sq. metre*
Seepage loss in lined channel = *0.01 cumec per million sq. metre of wetted perimeter.*
Saving in annual maintenance as a consequence to lining = *40%*
Life of lining = *40 years*
Rate of interest = *6 per cent*
Assume additional suitable data, if required. (20)

[**Note.** Please refer solved Example 5.1. **This question was set from this book with slight modifications in numerical data values.**]

Q. 4. *(a) Explain how maximum intensity-duration frequency curve is obtained.* (12)

(b) Define :

(i) Maximum Probable Flood

(ii) Standard Project Flood (10)

(c) Estimate the average depth of precipitation from depth area curve, that may be expected over an area of 2400 sq km, due to the storm of 27th Sept. 1978, lasting for 24 hours, assuming the storm centre to be located at the centre of the area. The isohyetal map for the storm gave the areas enclosed between different isohyets as follows :

Isohyetal in mm	*21*	*20*	*19*	*18*	*17*	*16*	*15*	*14*	*13*	*12*
Enclosed area in sq km	*543*	*1345*	*2030*	*2545*	*2955*	*3280*	*3535*	*3710*	*3880*	*3915*

Hence, determine the depth of rainfall, due to the storm that may be expected to be recorded by a raingauge placed at the storm centre. (18)

[**Note.** Please refer Example 7.7].

Year 1987 *(Engineering Services)*

Q. 5. (*a*) A round crested spillway passes a design discharge of 1 m^3/sec per metre length. The coefficient of discharge may be taken as $C_d = 0.7$. If the height of the crest above the downstream stilling basin floor level is 10 m, design the

(*i*) depth, and

(*ii*) length of the stilling basin. Depth of flow in the stream on the downstream of the spillway is 1 m at the design discharge of 1 m^3/sec. Enquire if the bed of the stilling basin has to be depressed. (20)

Q. 6. (*b*) A railway embankment is 16 m wide with side slopes 2 : 1. Assume the ground to be level in the direction transverse to the centre line. Calculate the volume contained in a length of 100 m, the centre heights at 20 m intervals being in m : 2.0, 4.5, 4.0, 3.5, 2.5, 1.5. Use Trapezoidal formula. (20)

Year 1988 *(Engineering Services)*

Q. 1. *(a) A certain stretch of lined trapezoidal channel has one vertical side wall and the other 45° sloping wall. If it is to deliver water at 30 m^3/sec with a velocity of 1 m/sec, compute bed width and flow depth for minimum lining area.* (10)

Q. 6. *(a) Define unit hydrograph.*

Ordinates of 8 hour unit hydrograph for a drainage basin are given below. Obtain a 24 hour unit hydrograph by tabulation method. Neatly sketch it.

Time hours (1)	Ordinates of 8 hr unit hydrograph (2)	Time hours (contd.) (1)	Ordinates of 8 hr unit hydrograph (contd.) (2)
0	0.00	48	57.00
4	5.50	52	42.00
8	13.50	56	41.00
12	26.50	60	22.00
16	45.00	64	14.00
20	82.00	68	9.50
24	162.00	72	6.60
28	240.00	76	4.00
32	231.00	80	2.00
36	165.00	84	1.00
40	112.00	88	0.00
44	79.00		

(40)

[**Note.** Please refer Example 7.34].

Year 1989 *(Engineering Services)*

Q. 2. *For use in Hargreaves method, in respect of Group D wheat crop, the following K values are suggested :*

% age of crop growing season	0	5	10	15	20	25	30	35	40	45	50
K	0.08	0.08	0.15	0.19	0.27	0.33	0.40	0.46	0.52	0.58	0.65

% age of crop growing season	55	60	65	70	75	80	85	90	95	100
K	0.71	0.77	0.82	0.88	0.90	0.90	0.80	0.70	0.60	0.60

Wheat is grown between 1 November to 15 March. The effective rainfall in the individual months is taken as follows :

Nov.–0.5 cm ; Dec.–1.6 cm; Jan–3.2 cm; Feb.–2.7 cm ; March–Nil. Percolation losses are nil throughout, Field Irrigation Efficiency is 70% and Gross Irrigation Efficiency is 75%. The mean class A pan evaporation through the respective months are:

Nov.–13 cm; Dec.–11 cm; Jan.–7 cm; Feb.–15 cm; March–10cm. Compute the gross irrigation requirement for the whole crop period including 7 cm of net pre-sowing requirement. (40)

[**Hint.** Follow the sample calculations given for wheat crop in table 2.14, and compute G.I.R. as equal to 55.02 cm, as per table given on next page].

Solution Table for Q. 2 of 1989 (Engg. Services) : *Growth period* = 1 Nov. to 15 March (135 days)

Dates	*No. of days upto mid point of interval from start*	*%age of growing season* col. (2) $\times \frac{100}{135}$	*Pan evaporation* (E_p) *in cm*	*Coefficient (K) interpolated values for %age values of col. (3) from given values*	$E_t = KE_p = C_u$ *i.e. col. (4) × col. (5); consumptive use in cm*	R_e = *Effective rainfall in cm (given)*	*Net irr. requirement* NIR = $C_u - R_e$ = col. (6) – col. (7)
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
Nov. 1—30	15	11	13	0.16	2.08	0.50	1.58
Dec. 1—31	46	34	11	0.46	5.06	1.60	3.46
Jan. 1—31	77	57	9	0.73	6.57	3.20	3.37
Feb. 1—28	106	78	15	0.90	13.50	2.70	10.80
March 1—15	128	95	10	0.60	6.00	Nil	6.0
					Z = 33.21 cm	8.00 cm	25.21 cm

N.I.R. = 25.21 cm

$$\text{F.I.R.} = \frac{N.I.R.}{\eta_f} = \frac{25.21}{0.70} = 36.01 \text{ cm}$$

$$\text{G.I.R.} = \frac{F.I.R.}{\eta_c} = \frac{36.01}{0.75} = 48.02 \text{ cm}$$

Net presowing requirement before 1 Nov. = 7 cm.

$\therefore$ Total gross irrigation requirement = 48.02 + 7 = 55.02 cm. **Ans.**

Q. 8. *(b) A 2 hour unit hydrograph in a rather steep catchment is given below :*

Time, hour	*0*	*2*	*4*	*6*	*8*	*10*	*12*
Discharge, 100 m^3/s	*0*	*0.54*	*1.75*	*1.27*	*0.58*	*0.25*	*0*

Compute the 1 hour unit hydrograph for the catchment. (25)

[**Hint.** Please refer Solved Example 7.51].

Year 1990 *(Engineering Services)*

Q. 1. *(c) Given that the unit discharge in a rectangular channel is 18 m^3/sec and that the head loss across a hydraulic jump that forms in this channel is 1.1 m, estimate the prejump and post jump depths.* (15)

Solution. Using eqn. (10.5), and (10.12), we have

$$y_1 y_2 (y_1 + y_2) = \frac{2q^2}{g} = \frac{2 \times 18^2}{9.81} = 66.06 \quad ...(i)$$

[q = unit discharge *i.e.*, discharge per unit width]

and $$H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2}$$

where $H_L = 1.1$ m (given)

$$\therefore \quad (y_2 - y_1)^3 = 4.4\, y_1 y_2 \quad ...(ii)$$

Putting $y_2 = m y_1$ in (*ii*), we get

$$(m-1)^3 \cdot y_1^3 = 4.4 \cdot m y_1^2$$

$$\therefore \quad y_1 = \frac{4.4m}{(m-1)^3} \quad ...(iii)$$

Putting $y_2 = m y_1$ and this value of y_1 in (*i*), we get

$$y_1 \cdot m \cdot y_1 (y_1 + m \cdot y_1) = 66.06$$

or $$m (m+1) \cdot y_1^3 = 66.06$$

or $$m (m+1) \left[\frac{4.4m}{(m-1)^3}\right]^3 = 66.06$$

or $$\frac{m (m+1) (m)^3}{(m-1)^9} = \frac{66.06}{4.4^3}$$

or $$\frac{m^4 (m+1)}{(m-1)^9} = 0.775$$

Solving by hit and trial, we get

$$m = 2.93$$

Substituting it in (*iii*), we get

$$y_1 = \frac{4.4 \times 2.93}{(1.93)^3} = \mathbf{1.79\ m}$$

$$\therefore \quad y_2 = 2.93 \times 1.79 = \mathbf{5.25\ m}$$

Ans.

Q. 2. *Work out the irrigation schedule based on the soil moisture concept, given the following information. Also extract the data on the total depth of irrigation water required and the respective dates of irrigation water supply :*

(a) The crop is grown in an appropriate soil with no restrictive layers within the top 1.5 m depth of soil.

(b) Normal root zone depth of the crop is 1.2 m.

(c) Bulk density of soil is 1.35.

(d) Field capacity is 18% and permanent wilting point is 7%.

(e) Moisture level in the soil is to be maintained at not less than one-third of available retention. Irrigation will then be done over a duration of 2 days at a uniform rate of supply and at a uniform rate of advance to fully and just compensate for the depletion.

(f) No extra water is ever required for leaching.

(g) Sowing is done on 1 November when the soil moisture is left just at field capacity in the entire root zone.

(h) For the crop, at the location, the average evapotranspiration rates are :

1 Nov. — 30 Nov.	*:*	*1.1 mm/day*
1 Dec. — 31 Dec.	*:*	*1.7 mm/day*
1 Jan. — 31 Jan.	*:*	*2.4 mm/day*
1 Feb. — 28 Feb.	*:*	*1.5 mm/day*
1 March—25 March	*:*	*3.5 mm/day*

(i) Harvesting is done on or after 26 March.

(j) There is expected an effective rainfall of 24 mm during 4 January to 19 January, both days inclusive with uniform intensity.

(k) By the end of the crop growth season, only the minimum water needed to be left unused in the root zone.

[**Hint.** Pl see Solved Example 2.16]

Q. 3. *(c) An equipment used in construction activity costs Rs. 40,000 at first purchase. The average annual expenditure thereon for OMR is estimated to be Rs. 9,000 for the next three years for which funds be procured at the beginning of each year. At the end of the third year, it can be scrapped at a then value of Rs. 10,000. If the acceptable rate of return is 15% per annum, what should be the annual income by this machine as reckoned at the end of each year ?*

[**Note.** Please refer Solved Example 20.2 in "*Hydrology and Water Resources Engineering*" (11th-2000 Edition) by the same author].

Q. 8. *(a) The average rainfall values over a catchment in three successive 2-hour intervals are 3, 5 and 2 cm, respectively. The ϕ index for the catchment is taken as 0.2 cm/hr. The 2 hr unit hydrograph ordinates are given in table. Base flow can be taken as 7 m^3/sec at the beginning of the storm linearly increasing to 9 m^3/sec at 2 hours after the direct runoff peak discharge, and later linearly decreasing to 8 m^3/sec at 4 hours after the end of the direct runoff. Compute the resulting flood hydrograph.*

Time, hours	0	2	4	6	8	10	12	14	16
Discharge, m^3/sec	0	10	40	70	50	20	10	6	0
			(Peak)						

(15)

[**Solution.** Unit hydrograph (U.H.) ordinates are tabulated in col. (2) of the solution table given below. The discharge hydrographs are now produced in three successions, by 3 successive storms, each containing effective rainfalls equal to $R-\phi$; *i.e.* $3-2\times0.2 = 2.6$ cm ; $5-2\times0.2 = 4.4$ cm ; and $2-2\times0.2 = 1.6$ cm, respectively. Hydrograph ordinates due to these three rains are computed by multiplying U.H. ordinates by each, because U.H. is produced by 1 cm rain. These values are worked out in col. (3), (4) and (5) respectively. All of them are then added in col. (6) to obtain direct runoff hydrograph (D.R.H.). Base flow is then computed, and added to obtain hydrograph of resulting discharge as given in table below..

Solution Table for Q. 8 (*a*) of Year 1990 (Engg. Services)

Time from start, hr	*U.H. ordinates*	*Discharge ordinates caused by 1st rain = col. (2) × 2.6*	*Discharge ordinates caused by 2nd rain (at 2 hr lag) = col. (2) × 4.4*	*Discharge ordinates caused by 3rd rain = col. (2) × 1.6 at 4 hr lag*	*Total D.R.H. = col. (3) + (4) + (5)*	*Base flow*	*Total flow ordinates in m^3/sec = col. (6) + (7)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*
0	0	0	—	—	0	7	7.0
2	10	26	0	—	26	7.4	33.4
4	40	104	44	0	148	7.8	155.8
6	70	182	176	16	374	8.2	382.2
8	50	130	308	64	502	8.6	510.6*
10	20	52	220	112	384	9	393.0
12	10	26	88	80	194	8.85	202.85
14	6	6	44	32	82	8.71	90.71
16	0	0	26.4	16	42.4	8.57	50.97
18	—	—	0	9.6	9.6	8.43	18.03
20	—	—	—	0	0	8.29	8.29
22	—	—	—	—	—	8.15	8.15
24	—	—	—	—	—	8	8.0

* Peak = 510.6 m^3/sec.

Year 1991 (*Engineering Services*)

Q. 5. *(a) Enumerate the assumptions on which Dupuit-Theim theory of ground water flow is based and state the range of validity of the law governing the flow. Define the following terms :*

(i) Storage coefficient, (ii) Specific yield, (iii) Specific retention, (iv) Porosity, stating clearly the relationship which exists among some of these terms. (20)

Year 1992 (*Engineering Services*)

Q. 4. *(b) The following data pertains to healthy growth of a crop :-*

(*i*) *Field capacity of soil* = *30%*

(*ii*) *Permanent wilting percentage* = *11%*

(*iii*) *Density of soil* = *1300 kg/m*^{3}

(*iv*) *Effective depth of root zone* = *700 mm*

(*v*) *Daily consumptive use of water for the given crop* = *12 mm*

For healthy growth, moisture content must not fall below 25% of the water holding capacity between the field capacity and the permanent wilting point. Determine the watering interval in days. (20)

[**Solution.** Max. water holding capacity of soil

= Field capacity m.c. – Permanent wilting pt. m.c.
= 30% – 11% = 19%

25% of Max. holding capacity = 0.25 × 19% = 4.75%

The left out available m.c. must at no stage be allowed to fall below this value and watering should be applied as soon as the left out m.c. falls to such a low level. Hence lower limit to which m.c. can be allowed to fall = 4.75% above wilting point m.c. (at which plants fade away) = 4.75% + 11% = 15.75%.

In other words, the m.c. will be allowed to vary between 30% and 15.75%. Water depth stored between these two limits

$$= \frac{\gamma d}{w}[30\% - 15.75\%]$$

$$= \frac{1300}{1000} \times 0.7\,[0.30 - 0.1575]$$

$$= 0.1297 = 12.97 \text{ cm.}$$

Since consumptive use is 12 mm/day ;

12 mm of water is consumed in = 1 day

$\therefore$ 12.97 cm of water will be consumed in

$$= \frac{1}{12} \times (12.97 \times 10) = 10.8 \text{ days.}$$

The watering should, therefore, be applied after every 10 days.

Note. 10.8 days cannot be rounded to 11 days, as otherwise the left out m.c. will fall down below the minimum required. **Ans.**]

Q. 7. *(b) A sluice across a channel 6 metre wide discharges a stream 1 meter deep. What is the flow rate when the depth upstream of the sluice is 7 metre ? On the downstream side, concrete blocks have been placed to create conditions for hydraulic jump to occur. Determine the force on the blocks if the downstream depth is 3 metre.* (15)

[**Solution.** Discharge through the sluice gate

$$Q = C_d (6 \times 1) \sqrt{2gH}$$

as width of channel = 6 m
area of flow = 6 m × 1 m

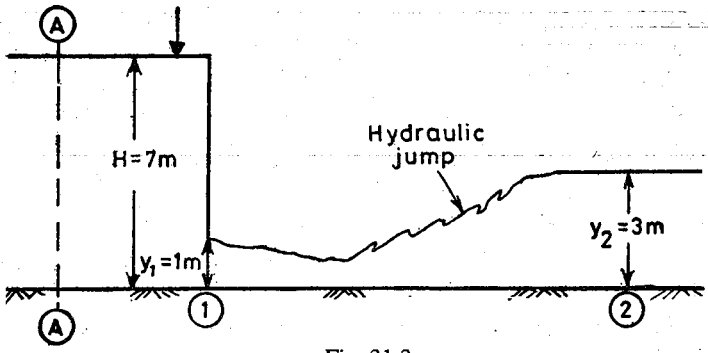


Fig. 31.3

where $C_d = C_c \cdot C_v$

Assuming $C_v = 0.95$

$C_c = 0.62$

$C_d = C_v \cdot C_c = 0.59$

$$Q = 0.59 \times 6 \times 1 \times \sqrt{2 \times 9.81 \times 7}$$

$$= 41.49 \text{ m}^3/\text{s} \quad \textbf{Ans.}$$

(*b*) Force acting on concrete blocks is given by the rate of change of momentum per unit channel width.

$$= \left(\frac{w}{g}\right) q (V_1 - V_2) \text{ kg}$$

where $q = \frac{Q}{6} = \frac{41.49}{6} = 6.91 \text{ m}^2/\text{s}$

$$V_1 = \frac{q}{C_c \cdot y_1} = \frac{6.91}{0.95 \times 1} \text{ m/s}$$

$$= 7.28 \text{ m/s}$$

$$V_2 = \frac{q}{y_2} = \frac{6.91}{3} = 2.3 \text{ m/s}$$

$$= \frac{1000 \times 6.91}{9.81} (7.28 - 2.3)$$

$$= 3504 \text{ kg} = 3504 \times 9.81 \text{ Newton} = 34.37 \text{ kN}. \quad \textbf{Ans.}]$$

Year 1993 *(Engineering Services)*

Q. 1. *(a) A water course commands an irrigated area 1000 hectares. The intensity of irrigation of rice in this area is 70%. The transplantation of rice crop takes 15 days and during the transplantation period, the total depth of water required by the crop on the field is 500 mm. During the transplantation period, the useful rain falling on the field is 120 mm. Find the duty of irrigation water for the crop on the field during transplantation at the head of the field, and also at the head of the water course*

assuming losses of water to be 20% in the water course. Also calculate the discharge required in the water course. (20)

[**Note :** Please follow exactly similar solved Example 3.5, and Compute : (*i*) 341 ha/cumec; (*ii*) 273 ha/cumec ; and (*iii*) 2.57 cumecs. **Ans.**]

Q. 2. *(c) A precipitation station X was inoperative for some time during which a storm occurred. At three stations A, B and C surrounding X, the total precipitation recorded during this storm are 75, 58 and 47 mm respectively. The normal annual precipitation amounts at stations X, A, B and C are respectively 757, 826, 618 and 482 mm. Estimate the storm precipitation for station X.* (10)

[**Note.** Please follow Exactly Similar Solved Example 7.1, and compute ppt. to be **71.2 mm. Ans.**]

Q. 3. *(c) During a recuperation test, the water in an open well was depressed by pumping by 2.5 m and it recuperated 1.8 m in 80 minutes. Calculate the yield from a well 4 m diameter under a depression head of 3 m.* (10)

[**Note.** Please follow exactly similar solved example 16.9, and compute yield as **10 l/sec. Ans.**]

Year 1994 *(Engineering Services)*

Q. 2. *(b) The information available from an isohyetal map of 1100 sq km basin is as follows :*

Zone	*Area* km^2	*Rain gauge station*	*Normal annual rainfall, cm*
I.	*85*	*A*	*120*
II.	*290*	*B* *C*	*95* *96*
III.	*395*	*D* *E* *F*	*60* *65* *70*
IV.	*230*	*G*	*45*
V.	*65*	*H*	*21*
VI.	*35*	—	—

How many additional rain gauge stations will be required if the desired limit of error in the mean value of rainfall is not to exceed 10 cm? Suggest how you propose to distribute these stations.

What factors will you consider in locating the additional rain gauge stations between different isohyetals ? (20)

[**Solution.** Follow Solved Example 7.3, and compute

$$\text{Mean rainfall } = \overline{P} = \frac{120+95+96+60+65+70+45+21}{8} = \frac{572}{8} = 71.5 \text{ cm}$$

$$\overline{P^2} = \frac{\Sigma p^2}{n}$$

$$= \frac{(120)^2+(95)^2+(96)^2+(60)^2+(65)^2+(70)^2+(45)^2+(21)^2}{8} = 5979$$

$$\sigma = \sqrt{\frac{n}{n-1}[\overline{P^2} - (\overline{P})^2]}$$

$$= \sqrt{\frac{8}{7}(5979 - (71.5)^2} = 31.47$$

$$C_v = \frac{100\sigma}{\overline{P}} = \frac{100 \times 31.47}{71.5} = 44.019$$

Optimum No. of Rain gauges

$$= N = \left(\frac{C_v}{E}\right)^2$$

where E = % error allowed in mean rainfall

$$= \frac{10 \text{ cm}}{\overline{P}} = \frac{10}{71.5} \times 100 = 13.986$$

$$\therefore \quad N = \left(\frac{44.019}{13.986}\right)^2 = 9.91 : \textbf{say 10}$$

Additional gauges required

$$= 10 - 8 = \textbf{2 No.} \quad \textbf{Ans.}]$$

One of these two additional gauges can be installed in Zone IV, which is having only one gauge, although covering an area of 230 km^2, *i.e.* about 20% of total catchment, and thus may need about 20% × 10 gauges = 2 gauges. The other additional gauge can be installed in Zone VI, which at present, is not having any gauge at all. **Ans.]**

Q. 5. *(c) The base period, intensity of irrigation, and duty of water for various crops under the canal system are given. Determine the reservoir capacity if the culturable commanded area is 40,000 hectares, canal losses are 25%, and reservoir losses are 15%.*

Crop	*Base period days*	*Duty of the water at the field hec/cumec*	*Intensity of irrigation %*
Wheat	*120*	*1800*	*20*
Sugarcane	*360*	*1700*	*20*
Cotton	*180*	*1400*	*10*
Rice	*120*	*800*	*15*
Vegetables	*120*	*700*	*15*

(15)

[Solution. Follow Example 3.6, and compute Areas of different crops under irrigation by multiplying *CCA* (40,000 hectares) with intensity of irrigation, as given in column (5) of solution table. Water required by each crop in cumecs is then determined in column (6) by dividing crop area in hec by duty in hec/cumec. The volume of water required by each crop in *B* days is finally computed in column (7) by multiplying base period in secs with water required in m^3/sec *i.e.* column (2) × 24 × 60 × 60 × column (6).

Solution Table for Q. 5 (c) of Year 1994 *(Engg. Services)*

Crop	*Base period (B) in days*	*Duty at the field in hec/cumec*	*I.I. %*	*Area of each crop under irrigation CCA × I.I. = 40,000 × Col. (4)*	*Water required by each crop in m^3/sec = Col. (5) / Col. (3)*	*Volume of water required in m^3 in (B) days = Col. (2) × 24 × 60 × 60 × Col. (6)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*
Wheat	120	1800	20	8,000	4.444	46.08 M-m^3
Sugarcane	360	1700	20	8,000	4.706	146.37 M-m^3
Cotton	180	1400	10	4,000	2.857	44.43 M-m^3
Rice	120	800	15	6,000	7.50	77.76 M-m^3
Vegetables	120	700	15	6,000	8.571	88.87 M-m^3
Σ			80%	32,000		403.51 M-m^3

Hence, total water required at the field

$= 403.51\ \text{Mm}^3$.

Canal losses $= 25\%$

∴ Water required at Canal Head

$$= \frac{403.51}{0.75}\ \text{Mm}^3 = 538.013\ \text{Mm}^3$$

Reservoir losses $= 15\%$

∴ Gross Reservoir Capacity $= \dfrac{538.013}{0.85}\ \text{Mm}^3 = 632.96\ \text{Mm}^3$

say **633 Mm^3** **Ans.]**

Year 1995 (*Engineering Services*)

Q. 1. *(b) Calculate the bed width for an irrigation channel to carry a discharge of 5 cumec. Side slopes of the channel are 1/2 horizontal to 1 vertical. Assume critical velocity ratio as 0.8. Take first trial depth of flow as 1 m. The channel has a bed slope of 0.2 m per kilometre. Using Kutter's equation $V = C \cdot \sqrt{R \cdot S}$, check the depth. The value of C for the given conditions is computed as 30. V is the mean velocity of flow in m/sec, and R is the hydraulic mean radius in metre.*

State the factors on which the value of C will depend. (15)

[**Hint.** Using $y = 1.0$ m (*assumed in 1st trial as given*), compute $V_0 = \mathbf{0.44\ m/s}$;

and $A = \dfrac{Q}{V_0} = \dfrac{5}{0.44} = 11.36 = \left(B + \dfrac{1.0}{2}\right)1.0$ or $B = 10.86$ m;

$P = 10.86 + \sqrt{5} \times 1.0 = 13.10$, $R = \dfrac{11.36}{13.10} = 0.867$.

Then $V = 30 \cdot \sqrt{0.867 \times \dfrac{0.2}{1000}} = \mathbf{0.395\ m/s}$. To reduce V_0 reduce y in second trial to about 0.7 m, when we get $V \approx V_0 \approx 0.35$ and $B = 20$ m. **Ans.**]

Q. 2. *(a) A stream has a width of 30 m, depth of 3 m and a mean velocity of 1.25 m/sec. Find the height of a weir to be built on the stream floor to raise the water level by 1 m. Assume value of discharge coefficient as 0.95.* (15)

[Hint. If x is the height of the weir, then head over the weir causing flow $= (4 - x)$, because D/s water depth is 3 m, and U/s water depth is 4m.

$$Q = \frac{2}{3} \cdot C_d \cdot \sqrt{2g} \cdot B \cdot h^{3/2} = 30 \times 3 \times 1.25$$

or $$\frac{2}{3} \times 0.95 \sqrt{2 \times 9.81} \times 30 \times (4 - x)^{3/2} = 30 \times 3 \times 1.25$$

or $x = \mathbf{2.79\ m}$ **Ans.]**

Q. 3. *(b) An aquifer of 20 m average thickness is overlain by an impermeable layer of 30 m thickness. A test well of 0.5 m diameter and two observation wells at a distance of 10 m and 60 m from the test well are drilled through the aquifer. After pumping at a rate of 0.1 m^3/sec for a long time, the following drawdowns are stabilised in these wells : first observation well, 4 m, second observation well 3 m. Show the arrangement in a diagram. Determine the coefficient of permeability and drawdown in the test well. State the validity of Darcy's Law.* (15)

[Hint. Solve the question using Thiem's formula for confined aquifers, and determine $K = \mathbf{1.425 \times 10^{-3}\ m/sec}$ **Ans.**

Then use r_w in place of r_1 and s_w in place of s_1 in this formula and compute $s_w = \mathbf{6.059\ m.}$ **Ans.]**

Q. 5. *(c) A farmer wishes to have his own pump set for the following cropping pattern to be followed in five hectares of his land. Calculate the right size of the centrifugal pump he should have, litres/sec.*

Season	*Crop*	*Area to be irrigated, ha*	*Intensity of irrigation, cm*	*Rotation period, days*
Rabi	*Wheat*	*2.0*	*7.5*	*12*
	Cotton	*0.4*	*7.5*	*20*
	Vegetables	*0.4*	*7.5*	*10*
	Mustard	*2.2*	*5.0*	*40*

For each crop, duration of pumping hours per day is 10. (10)

[Hint. It can be seen from the given data that the water is required for different crops as below :

(i) for wheat $= (2 \times 10^4\ m^2) \times \frac{7.5}{100}\ m = 300\ m^3$ @ every 12th day in 10 hrs.

(ii) for cotton $= (0.4 \times 10^4\ m^2 \times \frac{7.5}{100}\ m = 300\ m^3$ @ every 20th day in 10 hrs.

(iii) for vegetables $= (0.4 \times 10^4\ m^2) \left(\frac{7.5}{100}\ m \right) = 300\ m^3$ @ every 10th day in 10 hrs.

(*iv*) *for mustard* $(2.2\times10^4\ \text{m}^2)\left(\frac{5}{100}\ \text{m}\right)=1100\ \text{m}^3$ @ every 40th day in 10 hrs.

This shows that maximum water is required for wheat, *i.e.* $1500\ \text{m}^3$ in 10 hrs.

$\therefore$ Water required in l/s to be pumped

$$=\frac{(1500\times1000)\ l}{(10\times3600)\ \text{sec}}=\mathbf{41.67\ l/s}\quad\textbf{Ans.}$$

Q. 7. *(b) A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a batter of 1 : 10. Calculate (i) the factor of safety against overturning; (ii) Factor of safety against sliding; (iii) Shear friction factor. Assume coefficient of friction as 0.75. Unit weight of masonry as 2240 kg/cu.m. Permissible shear stress of joint = 14 kg/cm^2. Based on the results give your remarks.* (15)

[**Hint.** Draw the section of the dam, and compute

$\Sigma V = 103.6\ \text{t}\ ;\ \Sigma H = 50\ \text{t}$

$\Sigma M_V = 528\ \text{tm}\ ;\ \Sigma M_H = 166.5\ \text{t.m.}$

$\therefore$ F.S. against overturning $=\dfrac{\Sigma M_V}{\Sigma M_H}=\dfrac{528}{166.5}=\mathbf{3.17}$ **Ans.**

F.S. against sliding $=\dfrac{\mu\cdot\Sigma V}{\Sigma H}=\dfrac{0.75\times103.6}{50}=\mathbf{1.55}$ **Ans.** (> 1 ; OK)

$$\text{S.F.F.}=\frac{\mu\Sigma V\cdot B\cdot q}{\Sigma H}=\frac{0.75\times103.6+8.25\times14}{50}$$

$=\mathbf{3.86}$ **Ans.** (> 3–5 ; OK)

The structure is safe. **Ans.**]

Year 1996 (*Engineering Services*)

Q.1. *(a) For a date of maximum recorded flood of a river, the mean and standard deviation are 4200 m^3/s and 1705 m^3/s, respectively. Using Gumbel's extreme value distribution, estimate the return period of a design flood of 9500 m^3/s. Assume an infinite sample size.* (20)

[**Hint.** Use eqn = (7.142) as :

$$X=\overline{X}+k\cdot\sigma$$

where $\overline{X}=4200\ \text{m}^3/\text{s}$

$\sigma=1705\ \text{m}^3/\text{s}$

$X=9500\ \text{m}^3/\text{s}$

$\therefore\quad K=3.1085$

Now, use eqn. (7.148) as :

$$K=\frac{y_{(T)}-0.577}{1.2825}$$ for infinite sample size as given *i.e.* $N\to\alpha$

$\therefore\quad y_{(T)}=4.5637$

Now, use eqn. (7.146) as :

$$y_{(T)} = -\left[\ln \ln \frac{T}{T-1}\right]$$

$$\therefore \quad (-)\,4.5637 = \left[\ln \ln \frac{T}{T-1}\right]$$

or $$\frac{T}{T-1} = 1.0104$$

or $$\frac{T-1}{T} = 1 - \frac{1}{T} = 0.9896$$

or $$T = 96.43$$

Hence, the return period of design flood of 9500 m^3/s = **96.43 Yrs. Ans.**]

Q.2. *(b) The following is the set of observed data for successive 15 minute period of 105 minutes storm in a catchment.*

Duration (Min)	*Rainfall cm/hr.*
15	*2.0*
30	*2.0*
45	*8.0*
60	*7.0*
75	*1.25*
90	*1.25*
105	*4.5*

If the value of ϕ_{index} is 3.0 cm/hr, estimate the net runoff, the total rainfall and the value of W_{index} (20)

[**Hint.** Follow exactly 'similar solved example 7.24 and compute :

Total runoff $(Q) = (8-3)\frac{15}{60} + (7-3)\frac{15}{60} + (4.5-3)\frac{15}{60}$

or $$Q = \frac{5}{4} + \frac{4}{4} + \frac{1.5}{4} = \frac{10.5}{4} = \mathbf{2.625\ cm} \quad \textbf{Ans.}$$

Total rainfall = P

$$= 2 \times \frac{15}{60} + 2 \times \frac{15}{60} + 8 \times \frac{15}{60} + 7 \times \frac{5}{60} + 1.25 \times \frac{15}{60} + 1.25 \times \frac{15}{60} + 4.5 \times \frac{15}{60}$$

$$= 26 \times \frac{1}{4} = \mathbf{6.5\ cm} \quad \textbf{Ans.}$$

$$W_{index} = \frac{P-Q}{Tr} = \frac{6.5 - 2.625}{\left(\frac{105}{60}\right)} = \mathbf{2.214\ cm/hr}\ \textbf{Ans.}$$

]

Q.3. *(a) During a recuperation test, the water in an open well was depressed by pumping by 2.1 m and it recuperated by 1.6 m in 90 minutes. Find the diameter of a well to yield 10 litre/sec under a depression head of 2m.* (20)

[Hint. Follow exactly similar solved example 16.10, and compute :

$$\frac{C'}{A} = \frac{2.3}{T} \log \frac{s_1}{s_2} \qquad \text{(Eqn 16.39)}$$

$$= \frac{2.3}{90 \times 60} \log \frac{2.1}{2.1 - 1.6}$$

$$= 2.64 \times 10^{-4} \text{ m}^3/\text{sec}/\text{m}^2/\text{m}$$

Now use eqn. (16.40) as :

$$Q = \left(\frac{C'}{A}\right) \cdot A \,.\, s$$

$$\therefore \quad \frac{10}{1000} \text{ m}^3/\text{s} = (2.654 \times 10^{-4}) \times A \times 2$$

or $A = 18.83$

$\therefore$ diameter of well giving 10 $l/s = d =$ **4.9 m Ans.]**

Q 5. *(b) Enumerate the various types of energy dissipation devices which may be recommended below spillway in relation to the relative position of tail water rating curve and jump height rating curve.* (30)

[Hint. See article 21.6]

Q. 6. *(a) Design a lined canal to carry 100 m^3/s on a slope of 1 in 2500. The maximum permissible velocity is 2 m/s, $n = 0.013$ in Manning's formula, and the side slope = 1.25 H : 1.0 V.* (20)

[Hint. Follow exactly similar solved example 5.4 and compute.

$$2 = \frac{1}{0.013} \times R^{2/3} \cdot \frac{1}{\sqrt{2500}}$$

or $R = 1.482 \text{ m}$ ...(*i*)

$\cot \theta = 1.25$

$\theta = 0.675$ radians

From eqn = (5.8) ; $A = y\,(B + 1.925y) = \dfrac{Q}{V} = \dfrac{100}{2} = 50$

or $(B + 1925y) = \dfrac{50}{y}$

or $B = \left(\dfrac{50}{y} - 1.925y\right)$ ...(*ii*)

From eq. 5.9 ; $P = B + 3.85\,y$

$$R = \frac{A}{P} = \frac{50}{(B + 3.85\,y)} = 1.482 \qquad \text{...(iii)}$$

$\therefore \quad B + 3.85y = 33.74$

or $B = 33.74 - 3.85y$ ...(*iv*)

Equating (*ii*) and (*iii*)

$$\frac{50}{y} - 1.925y = 33.74 - 3.85y$$

or $$50 - 1.925y^2 = 33.74y - 3.85y^2$$

$$1.925y^2 - 33.74y + 50 = 0$$

or $$y = \frac{33.74 \pm \sqrt{33.74^2 - 4 \times 50 \times 1.925}}{2 \times 1.925} = \frac{33.74 \pm 27.45}{3.85}$$

$$\left.\begin{array}{l} y = 1.634\text{ m} \\ B = 27.45\text{ m} \end{array}\right] \quad \textbf{Ans.}$$

Q.8. *(a) Calculate the seepage through an earthen dam resting on an impervious foundation. The relevant data are given below :*

Height of the dam	= 60.0 m
Upstream slope	= 2.75 : 1 (H : *V*)
Downstream slope	= 2.50 : 1 (H : *V*)
Freboard	= 2.5 m
Crest width	= 8.0 m
Length of drainage blanket	= 120.0 m

Coefficient of permeability of the embankment material in

X-direction = 8×10^{-7} m/s

Y-direction = 2×10^{-7} m/s

(20)

[Hint. As a matter of fact such problems have generally been solved in "Soil Mechanics", and hence the students will have to refer and study article 7.7.1.1 in authors another book titled "Geotech Engineering – Soil Mechanics and Foundation Engineering – 3rd 1999 edition)", before attempting to solve this question.

According to the theory explained in that article, we have by eqn. (7.30)

q = discharge through dam = K.S.

where $K = \sqrt{K_H \cdot K_V}$

S = Focal distance of base parabola forming the top seepage line in the dam body.

S is further given by eq. (7.28) as

S = Focal distance w.r. Fig. 7.22 in "Soil Mechanics"

$= \sqrt{b^2 + H^2} - b$

where H = Max. water depth = 60 – 2.5 = 57.5 m

= 47 + 8 + 30 (Please see Fig. 31.4)

= 85 m

$\therefore\ S = \sqrt{85^2 + 57.5^2} - 85 = 17.62$ m

$q = \sqrt{K_H \cdot K_v} \cdot S$

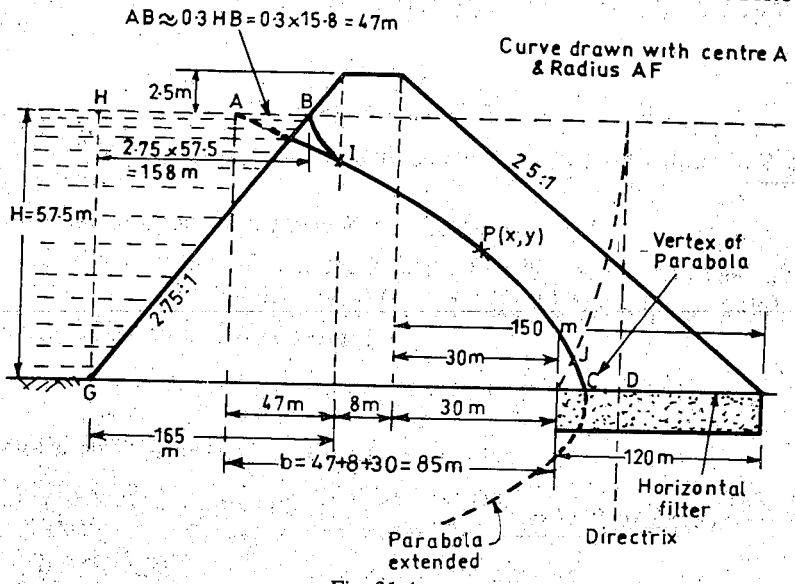


Fig. 31.4.

$= \sqrt{8 \times 10^{-7} \times 2 \times 10^{-7}} \times 17.62\ m^3/sec$ per m width of dam

$= 4 \times 10^{-7} \times 17.62\ m^3/s.m$

$= 70.48 \times 10^{-7}\ m^3/sec.m = 0.610\ m^3/day.\ m$

= 610 litres per day per *m* width of dam. **Ans.**

Year 1997 (*Engineering Services*)

Q. 1. *(a) A large sample of peak floods data was available for a river. Flood frequency computations using Gumbel's method, yield the following results.*

Return Period T (Years)	*Peak Flood m^3/sec*
50	*30,800*
100	*36,300*

Estimate the magnitude of a flood for this river with a return period of 200 years. (15)

[Hint. Follow exactly similar solved Example 7.57 and determine $X_{(200)} = 41780\ m^3/sec$ **Ans.]**

Q. 1. *(c) A most efficient trapezoidal section is required to give a maximum discharge of 21.5 m^3/s of water. The slope of the channel bottom is 1 in 2500. Taking C = 70 $m^{1/2}/s$ in Chezy's equation, determine the dimensions of the channel. Also determine the value of Manning's 'n' taking the value of velocity of flow as obtained for the channel by Chezy's equation.* (15)

[Hint. Pl refer Solved Example 4.5]

Q. 2. *(c) Show that the drawdown in the case of a partially penetrating well in a confined aquifer (well just penetrating to the top of an infinite porous medium) is given by the equation.*

$$S_w = \frac{Q_s}{2\pi \cdot r_w \cdot K}$$

where Q_w = discharge from well under steady conditions

r_w = radius of the well

K = Coefficient of permeability

S_w = drawdown at the well

Also find an expression for $\frac{Q_s}{Q_r}$

where Q_r = discharge from a fully penetrating well in confined aquifer under steady state (derivation for Q_r not required)

(10)

[**Hint.** Please refer article 16.13]

Q. 3. (*a*) Sugarcane (root zone depth 1.8 m) is grown in a particular area where the ground watertable is 2.0 m below ground. If the size of the soil pores is 0.08 mm in diameter and surface tension $\sigma = 0.054$ N/m, is the field water-logged ? If so, determine the vertical location of closed dreius below ground, spaced at 15 m. Take drainage coefficient as 0.116 *cumecs/km*^{2}, Coefficient of permeability as 10^{-6} m/s, and the impervious stratum to occur at 7.0 m below ground. Assume the capillary rise from ground watertable not to interfere with the root zone of plant. (15)

[**Hint.** Please refer Solved Example 6.5]

Q. 6. (*c*) The following figure gives profile of a gravity dam with reservoir level as shown. If the coefficient of friction is 0.75, is the dam safe against sliding ? Take weight density of concrete = 2.4 Tonnes/*m*^{3}.

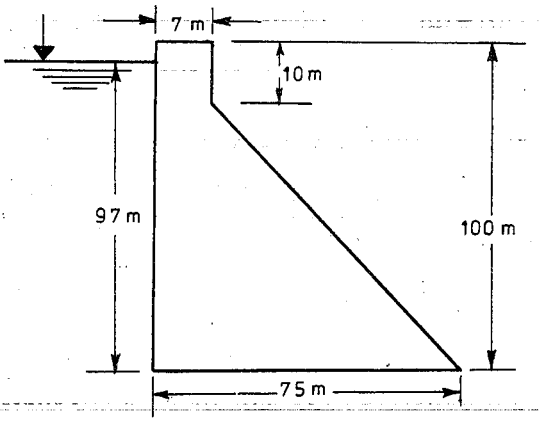


Fig. 31.5

(15)

Year 1998 (*Engineering Services*)

Q. 3. (*c*) What are the factor affecting evaporation from water surfaces ? Describe briefly a method of estimating the evaporation from weather data. (10)

Q. 6. *(c) Compute and draw the storm hyetograph and the intensity duration curve for the following storm (of a given-frequency) on a drainage basin.*

Duration (Minutes)	*Accumulated Precipitation (cm)*
0	–
30	*5.0*
60	*7.5*
90	*8.5*
120	*9.0*

(15)

[**Hint.** Follow exactly similar solved numerical Example 7.11]

Q. 7. (*a*) A soil 50 cm deep over rock has two horizons, the first being a fine sandy loam 20 cm thick and the second a clay loam 30 cm thick. The field capacity, wilting point and volume weight for the first horizon are 20 percent, 10% percent and 1.5 gm/cm^3, respectively. The corresponding values for the second horizon are 25 percent, 13 percent and 1.2 gm/cm^3. Determine the available moisture storage capacity of the soil profile. If consumptive use requirements of a crop in a particular season is 0.5 mm/day and the soil is initially at field capacity, how long will the crop survive without irrigation. (15)

[**Solution.**

(*i*) Moisture holding capacity of first horizon (fine sandy loam of 20 cm thick)

$$= \frac{\gamma_d \cdot d}{\gamma_w} [F.C. - W.P]$$

$$= \frac{1.5}{1} \times 20 \text{ cm } [20\% - 10\%] = 3 \text{ cm} \quad ...(i)$$

(*ii*) Moisture holding capacity of 2nd horizon (clay loam of 30 cm depth)

$$= \frac{1.2}{1} \times 30 \text{ cm } [25\% - 13\%] = 4.32 \text{ cm} \quad ...(ii)$$

Total max available moisture *i.e.* storage capacity of the soil

$$= (i) + (ii)$$

$$= 3 \text{ cm} + 4.32 \text{ cm} = \mathbf{7.32 \text{ cm}} \quad \textbf{Ans.}$$

Consumptive use requirement per day

$$= 0.5 \text{ mm/day} = 0.05 \text{ cm/day}$$

Max. No. of days in which the entire moisture storage capacity will be utilised *i.e.* the absolute No. of days for which crop will survive

$$= \frac{\text{Moisture Storage capacity in cm}}{\text{Consumptive use in cm/d}}$$

$$= \frac{7.32 \text{ cm}}{0.05 \text{ cm/day}} = \mathbf{146.4 \text{ days.}} \quad \textbf{Ans.}$$

Q. 7. *(c) A vertical lift gate 5m × 2.5 m size weighing 0.5 Tonnes slides along guides (coefficient of friction 0.25) fitted on the side walls of an overflow spillway and its crest. What force will have to be exerted at the hoisting mechanism to lift the gate when the head of water over the crest is 2 m ?* (10)

[**Solution.** In article 24.14.3, while describing the vertical lift gate, we have clearly stated that when such a gate is sliding between groove guides not provided with rollers, force will be required to lift the gate at the hoisting mechanism for not only lifting the self wt. of the gate but also

to overcome the friction that will be developed between the gate and the d/s groove guides due to the hydrostatic force caused by the water standing against the gate

(i) self weight of gate = 0.5 tonne wt.

$$= 0.5 \times 9.81 \text{ kN}$$
$$= 4.91 \text{ kN} \quad ...(i)$$

(ii) Hydrostatic force acting on the gate of ht 2.5 m & 5 m span with 2 m water height

$$P_H = \frac{1}{2} \times (9.81 \times 2) \times 2 \text{ m} \times 5 \text{ m} = 98.1 \text{ kN}$$

$$\mu = 0.25$$

$\therefore$ Friction force (downward)

$$= \mu \cdot P_H = 0.25 \times 98.1 \text{ kN}$$
$$= 24.53 \text{ kN} \quad ...(ii)$$

Total Upward force required to lift the gate

$$= (i) + (ii) = 4.91 + 24.53 = 29.44 \text{ kN}.$$

Hence, the force required at hoisting to lift the gate

$$= 29.44 \text{ kN} = \frac{29.44}{9.81} \text{ t force} \quad \textbf{Ans.}$$

$$= \mathbf{3\ t \cdot force} \quad \textbf{Ans.}$$

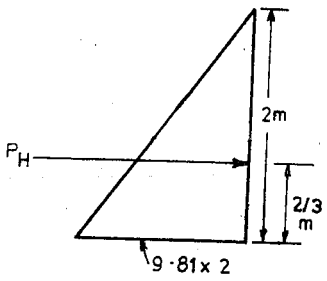


Fig. 31.6

Q. 8. *(a) An irrigation canal takes off from a perennial river. Sketch a typical layout of a diversion headworks. Indicate therein the various components of the headworks.* (15)

Year 1999 (Engineering Services)

Q. 4. *(a) The amounts of water flowing from a certain catchment area at the proposed dam site are as tabulated below :*

Month	*Inflow*
January	*2.83*
February	*4.25*
March	*5.66*
April	*18.40*
May	*22.64*
June	*22.64*
July	*19.81*
August	*8.49*
September	*7.10*
October	*7.10*
November	*5.66*
December	*5.66*

Determine :

(i) *the minimum capacity of reservoir if water is to be used to feed the turbines of hydropower plant at a uniform rate and no water is to be spilled over.*

(ii) *the initial storage required to maintain the uniform demand as above* (15)

[**Hint.** Follow exactly similar solved example 18.8, and compute

(i) Min capacity of reservoir = 40.08×10^5 cum
(ii) Initial storage reqd. = 19.82×10^5 cum.

] **Ans.**

Q. 5. *(a) A river discharges 1000 cum/sec of water at high flood level of RL. 103 m. A weir is constructed for flow diversion with a crest length of 255m and total length of concrete floor as 40m. The weir has to sustain the under-seepage at a maximum static head of 2.4m. The silt factor and the safe exit gradient for the river bed material are 1.1 and 1/6, respectively. Determine the depth of cut off required at the downstream end of the concrete floor. Take the level of downstream concrete floor as RL 100.0m. Check for exit gradient.* (15)

[**Solution.** High flood discharge = Q = 1000 m^3/s.

Length of weir = L = 255 m

$$q = \frac{Q}{L} = \frac{1000}{255} = 3.92 \text{ m}^2/\text{s}$$

f = 1.1 (given)

$$R = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{3.92^2}{1.1}\right)^{1/3} = 3.25 \text{ m}$$

Provide D/s cutoff up to 1.5 R below d/s water level.

1.5 R = 1.5 × 3.25 = 4.88 m

D/s W.L. = ?

U/s WL = U/s HFL = 103.00 m (given)

H = Max. Static head causing seepage = 2.4 m (given)

∴ D/s W.L. = U/s HFL − H = 103.0 − 2.4 = 100.6 m

Hence, R.L. of bottom of d/s cutoff = 100.6 − 4.88 = 95.72 m.

R.L. of D/s floor = 100.0 m (given)

∴ depth of d/s cutoff = 100.0 − 95.72 = 4.28 m.

Hence d = **4.28 m** **Ans.**

Exit gradient $G_E = \frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}}$

where $\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$

where $\alpha = \frac{b}{d} = \frac{\text{Length of weir floor}}{\text{depth of d/s cutoff}}$

where, b = 40 m (given)

d = 4.28 m (calculated above)

∴ $\alpha = \frac{40}{4.28} = 9.346$

∴ $\lambda = \frac{1+\sqrt{1+9.346^2}}{2} = 5.2$

∴ $G_E = \frac{2.4 \text{ m}}{4.28 \text{ m}}\frac{1}{\pi\sqrt{5.2}} = \mathbf{0.078}$ **Ans.**

$= \frac{1}{12.77} < \frac{1}{6}$ (Safe exit gradient)

Hence, the weir is safe from exit gradient considerations with bottom of d/s cut off at RL 95.72 m. **Ans.**

Q. 6. *(c) What are the assumptions and limitations of Sherman's Unit Hydrograph Theory ?* (10)

[**Hing.** Pl See article 7.45. 1].

Q. 8. (*b*). *A sandy loam soil holds water at 140 mm/m depth between field capacity and permanent wilting point. The root depth of the crop is 30 cm and the allowable depletion of water is 35%. The daily water use by the crop is 5 mm/day. The area to be irrigated is 60 ha and water can be diverted at 28 lps. The surface irrigation application efficiency is 40%. There are no rainfall and ground water contribution.*

Determine

(*i*) *allowable depletion depth between irrigations.*

(*ii*) *frequency of irrigation*

(*iii*) *net application depth of water*

(*iv*) *Volume of water required*

(*v*) *time to irrigate 4 ha plot* (15)

[**Hint:** Pl see Solved Example 2.16]

Year 2000 (*Engineering Services*)

Q.5(*a*). *The regression analysis of a 30-year flood data at a point on a river yielded sample mean* $\bar{x} = 1200\ m^3/sec$ *and standard deviation* $s_x = 650\ m^3/sec$. *For what discharge would you design the structure at this point to provide 95% assurance that the structure would not fail in the next 50 years ? Use Gumbel's method. The value of mean and standard deviation of the reduced variate for* $n = 30$ *are 0.53622 and 1.11238, respectively.* (15)

[**Solution.** Follow article 7.50 3.2 and solved examples 7.58 & 7.55 and compute as under:

$\bar{y}_n$ = reduced mean for $n = 30$

$= \mathbf{0.53622}$

s_n = reduced standard deviation for $n = 30$

$= \mathbf{1.11238}$

$\bar{X}$ *i.e.* $\bar{x} = 1200\ m^3/sec$. (given)

σ = standard deviation of variate x

$= 650\ m^3/sec$ (given under symbol s_x)

$X_{(T)}$ = Discharge for return period T years = ?

$T = ?$

Return period T has to be such that the structure does not fail in the next 50 years with 95% assurance. To find this value of T, please refer solved example 7.55, and compute as under :

The structure has not to fail in the next 50 years with 95% assurance means that it can tolerete a risk of 5% ; *i.e.* $R = 5\% = 0.05$ in 50 years (*i.e.* $n = 50$ years]

Using $R = [1 - (1 - p)^n]$, we get

where $n = 50$ years

$$0.05 = [1 - (1 - p)^{50}]$$

or $$(1 - p)^{50} = 1 - 0.05 = 0.95$$

or $$1 - p = (0.95)^{1/50} = (0.95)^{0.02} = 0.9989746$$

or $$p = 1 - 0.9989746 = 0.001025$$

$$\therefore \quad T = \frac{1}{p} = \frac{1}{0.001025} = \mathbf{975.29\ yrs.}$$

For this value of T, we have to determine $X_{(T)}$ as below

$$X_T = \bar{X} + K_{(T)}\,\sigma$$

where $K_{(T)} = \dfrac{y_{(T)} - \bar{y}_n}{s_n}$ (*i.e.* Eq. 7.148*a*)

For $T = 975.29$ year, $y_{(T)}$ is given by eqn. (7.146) as :

$$y_{(T)} = -\left[I_n \,.\, I_n \,.\, \frac{T}{T-1}\right]$$

$$y_{(975.29)} = -I_n \,.\, I_n \,.\left(\frac{975.29}{974.29}\right) = 6.882222$$

$$\therefore \quad K_{(975.29)} = \frac{6.882222 - 0.53622}{1.11238} = 5.70488$$

The expected peak discharge having frequency of 975.29 years is now given as

$$X = \bar{X} + K.\sigma$$

or $$X_{(979.29)} = 1200 + 5.70488 \times 650$$

$$= 1200 + 3708 = \mathbf{4908\ m^3/sec}\ \ \textbf{Ans.}$$

Q. 6(*a*). *In order to compute the flood discharge in a stream by the slope-area method, the following data have been obtained :*

	u/s Section	*Middle Section*	*d/s Section*
Area	108.6 m^2	103.1 m^2	99.8 m^2
Wetted Perimeter	65.3 m	60.7 m	59.4 m
Gauge Reading	316.8 m	—	316.55 m

Determine the flood discharge assuming Manning's n = 0.029 and length between d/s and u/s sections as 250 m (15)

[Hint. *Follow exactly similar solved numerical example 7.23.* The only point here is that the values of Area (A) and wetted perimeter (P) given here at mid section are not to be used any where, except to infer that the stream is a *gradually contracting* one, since the area of cross-section as well as wetted perimeter values are gradually decreasing. Additionally, in solved example 7.23, the values of hydraulic mean depths(R) were given ; here they can be worked out as $R = \dfrac{A}{P}$; otherwise this numerical example can be solved similarly, by assuming $h_e = K_e\left(\dfrac{V_2^2}{2g} - \dfrac{V_1^2}{2g}\right)$; where $K_e = 0.1$. The computed value of flood discharge initially works out to 159.89 m^3/sec. which on correction for velocity head & eddy loss h_e, works out to **154** m^3/sec **Ans.**]

Q. 7(*b*). *For border strip method of irrigation, determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube-well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5 cm/hr. and the average depth of flow on the field as 10 cm. Also determine the maximum area that can be irrigated from this tubewell.* (15)

[**Hint.** Refer solved example 1.1. **This numerical question was set from this book]**

Q. 8(*a*). *Fig given below shows the section of a gravity dam (non-overflow portion) built of concrete. Calculate the maximum vertical stresses at the heel and toe of the dam. Assume weight of concrete at 23.5 kN/m³. Neglect earthquake effects.*

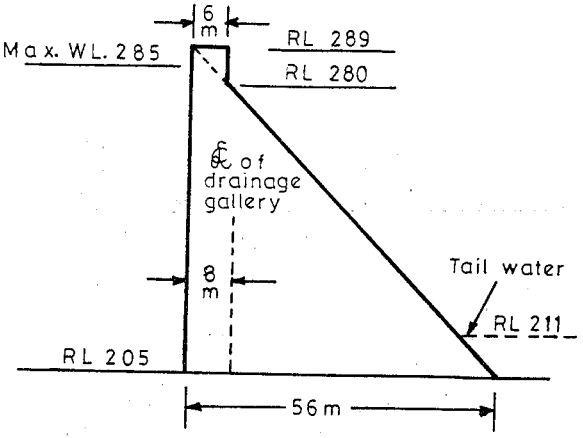


Fig. 31.7

[**Note.** Pl refer solved example 19.1 (*i*) **This numerical question was set from this book]**

Year 2001 (*Engineering Services*)

Q. 2. (*b*) *An unlined irrigation canal has its bed and sides composed of cohesionless material having mean diameter 6 mm. Angle of repose of the material is 40°. The bed width of the canal is 5 m and the side slope 1.5 (H) : 1 V. Determine the maximum discharge that can be admitted into the canal without any sediment movement. Longitudinal slope of the canal is 1 in 5000, and Manning's n = 0.025.* (15)

[**Solution.** Follow & refer solved Example 4.3, and compute as under :

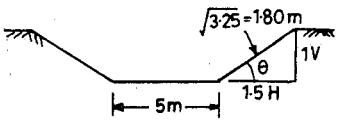


Fig. 31.8

$$\phi = 40° \text{ (given)} \qquad \tan\theta = \frac{1}{1.5}$$

$$\sin\phi = \sin 40° = 0.6428 \qquad \sin\theta = \frac{1}{\sqrt{3.25}} = \mathbf{0.5547}$$

$$\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \left[1 - \frac{(0.5547)^2}{(0.6428)^2}\right]^{1/2} = \sqrt{0.255} = 0.505.$$

$\therefore$ Min. shear stress required to dislodge the soil grain on side slope of canal is given by

$$\tau_c' = 0.505\, T_c$$

Hence, for stability, the shear stress actually going to be generated on slopes of the channel of given R & S must be less than or equal to $0.507\ \tau_c$; *i.e.*

$$\tau_0' \le 0.507\, \tau_c$$

The actual shear stress actually going to be generated on the side slopes of a channel of given R & S is given by Eqn. (4.17) as :

$$\tau_0 = 0.75\, \gamma_w \cdot R.S.$$

$\therefore$ $$0.75 \cdot \gamma_w \cdot R.S. \le 0.507 \cdot \tau_c$$

But $$\tau_c = \gamma_w \cdot R.S. = \gamma_w \cdot \frac{d}{11}$$

$\therefore$ $$0.75 \cdot \gamma_w \cdot R.S. \le 0.507\, \gamma_w \cdot \frac{d}{11}$$

or $$RS \le \frac{0.507}{0.75 \times 11} d \quad \text{or} \quad RS \le 0.0615\, d$$

where d = mean particle dia = 6 mm = 0.006 m

S = 1 in 5000

or $$RS \le 0.0615 \times 0.006 = 3.69 \times 10^{-4}$$

or $$R \times \frac{1}{5000} \le 3.69 \times 10^{-4} \quad \text{or} \quad R \le 1.845 \text{ m.}$$

using 20% factor of safety, we have

max. R = 1.47 m.

For 5 m wide trapezoidal channel with 1.5 (H) : 1 (V) side slopes, we have

$$R = \frac{A}{P} = \frac{(5 + 1.5y)\, y}{(5 + 2 \times 1.8y)} = \frac{(5 + 1.5y)\, y}{(5 + 3.6y)}$$

$\therefore$ $$1.47 = \frac{(5 + 1.5y)\, y}{5 + 3.6\, y}$$

Solve by Hit & Trial, and find

$$y \approx 2.3 \text{ m.}$$

$\therefore$ $$A = (5 + 1.5 \times 2.3)\, 2.3 = 19.44$$

$$P = (5 + 2 \times 1.8 \times 2.3) = 13.28$$

$$R = 1.46 \approx 1.47 \text{ (ok)}$$

$$Q_{max.} = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

$$= \frac{1}{0.025} \times 19.44 \times (1.46)^{2/3} \frac{1}{\sqrt{5000}} = \mathbf{14.15\ m^3/sec.} \quad \textbf{Ans.}$$

Q. 3. (*b*) *The following are the ordinates for a flood hydrograph resulting from an isolated storm of 6 hours duration.*

Time (hr.)	0	12	24	36	48	60	72	84	96
Ordinates of Flood hydrograph (cumecs)	5	15	40	80	60	50	25	15	5

Determine the ordinates of 1 cm-6 hr. unit hydrograph, if the catchment area is 450 sq. km. (15)

[**Hint :** Follow similar Solved Example 7.39, Assuming base flow = 5 m^3/s, list the ordinates of flood hydrograph as 0, 10, 35, 75, 55, 45, 20, 10 & 0. Compute vol. of water contained in this hydrograph as :

$$= \frac{12 \times 60 \times 60}{3}\left[\frac{0+0}{2} + 4\,(10+75+45+10) + 2\,(35+55+20)\right]$$

$$= 11.232 \times 10^6 \text{ m}^3.$$

Catchment area $= 450 \times 10^6 \text{ m}^2$

$\therefore$ Depth of water in the hydrograph $= \frac{11.232 \times 10^6}{450 \times 10^6}$ m = 2.5 cm.

Ordinates of U.H. (of 1 cm depth) can be obtained by dividing the above ordinates of flood hydrograph by 2.5. Hence, the ordinates of U.H. are : 0, 4, 14, 30, 22, 18, 8, 4, 0 **Ans.**]

Q. 4. (*c*). *A lined irrigation canal with trapezoidal cross-section has 5 m bed width, 2.5 m depth and 1.5 (H) : 1(V) side slope, longitudinal bed slope of the canal is 1 in 1000 and Manning's n = 0.016. What is the maximum carrying capacity of the canal ? What area of land in hectares the canal can irrigate if the crop has 150 mm field irrigation requirement in a core period of 10 days.* (10)

[Solution.

For a lined trapezoidal channel, corners are rounded, due to which A & P are computed by Eqn. 5.8 & 5.9 as :

$$A = y\,(B + y\,\theta + y \cot\theta)$$

$$P = B + 2y\,\theta + 2y \cot\theta$$

Here for 1.5 (H) : 1(V) side slopes, we have

$$\tan\theta = \frac{1}{1.5} = 0.8,$$

$$\cot\theta = 1.5, \quad \theta = 0.588 \text{ radians}$$

$$y = 2.5 \text{ m}, \quad B = 5 \text{ m}$$

$\therefore$ $$A = 2.5\,(5 + 2.5 \times 0.588 + 2.5 \times 1.5) = 25.55$$

$$P = (5 + 2 \times 2.5 \times 0.588 + 2 \times 2.5 \times 1.5) = 15.44$$

$$R = \frac{A}{P} = \frac{25.55}{15.44} = 1.655$$

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

$$= \frac{1}{0.016} \times 25.55\,(1.65)^{2/3} \frac{1}{\sqrt{1000}} = 70.65 \text{ m}^3/\text{s}$$

Hence, max. carrying capacity of channel = **70.65 m^3/sec. Ans.**

During core period of 10 days, vol. of water which can be supplied by the channel

$$= 70.65 \times (10 \times 24 \times 60 \times 60) \text{ m}^3 = 61.04 \times 10^6 \text{ m}^3.$$

Area which can be irrigated (A) × Depth of water required

= Vol. of water available

$\therefore \quad A \times 0.15 \text{ m} = 61.04 \times 10^6 \text{ m}^3$

or $$A = \frac{61.04 \times 10^6}{0.15} \text{ m}^2 = 407 \times 10^6 \text{ m}^2 = \textbf{407 sq. km Ans.}]$$

Q. 7 (*b*). *Following particulars were recorded from a barrage :*

(*i*) Maximum reservoir level = 212 m
(*ii*) Pond level = 211 m
(*iii*) Downstream high flood level in the river = 210 m
(*iv*) Maximum design flood discharge = 3500 m^3/s
(*v*) Crest level of the barrage = 207 m
(*vi*) Crest level of the head regulator = 208 m
(*vii*) Coefficient of discharge = 2.10 $m^{1/2}$/sec for barrage
= 1.50 $m^{1/2}$/sec for head regulator
(*viii*) River bed level = 205 m
(*ix*) Design discharge of main canal = 500 m^3/sec

Determine the number of gates required for the barrage and the head regulator if each gate has 10 m clear span. Neglect :

(1) end contractions due to piers and abutments ;

(2) velocity of approach.

If a stilling basin is provided downstream of the barrage for the energy dissipation, find the length and RL of the basin floor. Assume that the length of basin is 5 times the conjugate depth required for hydraulic jump. Neglect losses due to friction. (20)

Solution. (1) **Waterway for Barrage :**

High flood discharge of 3500 m^3/s has to pass through the gated openings provided at the barrage site, with the following data.

Max. water level at high flood U/S of barrage = 212.00 m
Crest level of barrage = 207.00 m
Opening available or Head causing flow above the crest of barrage
= 212.00 – 207.00 = 5.00 m.
(ignoring Vel. head)

Now, $Q = C \cdot L H^{3/2}$

where C = coeff. of discharge through the barrage
= 2.10 (given)
L = Effective waterway of barrage (openings)
H = 5.00 m

$\therefore \quad 3500 = 2.10 \times L \times (5)^{3/2}$

or $$L = \frac{3500}{2.1 \times (5)^{3/2}} = 149.1 \text{ m ; say } 150 \text{ m}$$

Effective water way = Total water way (ignoring end contractions)
Hence, provide 15 bays, each of 10 m clear span.
$\therefore$ **No. of gates required for barrage = 15 Ans.**

(2) **Water way for Canal Head Regulator.**

The discharge through canal regulator = 500 m^3/s
Coefficient of discharge for regulator opening = 1.5

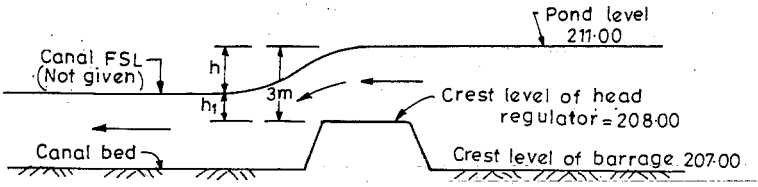


Fig. 31.9.

Head over the crest of regulator at pond Level = 211.00 – 208.00 = 3.00 m.

$$Q = C_1 \cdot L_1 \cdot H_1^{3/2}$$

$$\therefore \quad 500 = 1.5 \times L_1 \times (3)^{3/2}$$

or $$L_1 = \frac{500}{1.5 \times (3)^{3/2}} = 64 \text{ m.}$$

Provide 7 bays each of 10 m clear span, thereby providing 70 m water way.
No. of gates required for canal head regulator each of 10 m span = **7 Ans.**

(3) **Design of Stilling Basin.**

U/S HFL = 212.00

D/S HFL = 210.00

U/S TEL = 212.00 (ignoring vel. of approach)

D/S TEL = 210.00 (– do –)

H_L = 212.00 – 210.00 = **2 m**

q = Discharge intensity through barrage bays

= 3500/150 = **23.34** m^3/s/m.

E_{f_2} for H_L = 2 m & q = 23.34, from Blench curves – (Plate 10.1) is found to be = **7.2** m

∴ Level at which jump will form

$$= D/S\ TEL - E_{f_2} = 210.00 - 7.2 = \mathbf{202.8\ m}$$

y_2 corresponding to E_{f_2} of 7.2 m from plate 10.2 = 6.6 m.

Length of cistern = $5y_2 = 5 \times 6.6 = 33$ m.

Hence, provide 33 m long stilling basin at level say *202.6 m* (slightly lower than the calculated value of 202.8 m) as shown in Fig. 31.10 below.

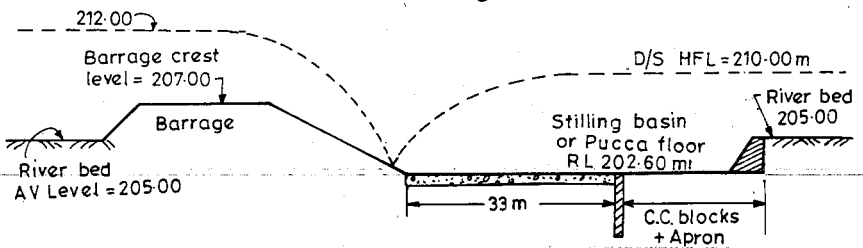


Fig. 31.10.

Alternatively, if Blench curves are not supplied in the Examination Hall, then we can compute the value of y_2 as follows :

$$y_c = \sqrt[3]{\frac{q^2}{g}} \qquad \text{(from Eqn. 10.3)}$$

$$= \sqrt[3]{\frac{23.3^2}{9.81}} = \mathbf{3.81\ m.}$$

Now let $Y = y_2/y_c$ (Eqn. 10.5)

and $Z = H_L/y_c$ (Eqn. 10.6)

Compute Value of $Z = \dfrac{2\text{ m}}{3.81\text{ m}} = \mathbf{0.5248}$

Use Eqns. 10.9 & 10.10 to compute the value of Y for known value of Z.

Here $Z < 1$, hence Eqn. (10.9) is applicable

$\therefore$ $Y = 1 + 0.93556\,Z^{0.368}$ (for $Z < 1$)

or $Y = 1 + 0.93556\,(0.5248)^{0.368} = \mathbf{1.738}$

$\therefore$ $Y = y_2/y_c = 1.738$

or $y_2 = 1.738\,y_c = 1.738 \times 3.81\text{ m} = \mathbf{6.6\ m.}$

$5y_2 = \text{Length of cistern} = 5 \times 6.6 = \mathbf{33\ m.}$ **Ans.**

Also, E_{f_2} can be calculated by Eq. (10.13) as :

$$\frac{E_{f_2}}{y_c} = Y + \frac{1}{2Y^2} = 1.738 + \frac{1}{2\,.\,(1.738)^2}$$

or $E_{f_2} = 1.904 \times 3.81 = \mathbf{7.25\ m}$

Level at which jump will form

$= D/S\ TEL - E_{f_2}$ $D/S\ TEL = D/S\ HFL$

$= D/S\ HFL - E_{f_2}$ (on neglecting vel. of approach)

$= 210.00 - 7.25 = 202.75$ m

Provide cistern at RL **202.6 m** **Ans.**

Q. 7. (*c*) *During re-cuperation test, the water level in an open well was depressed by pumping up to 2.5 m. The water level was raised by 1 m within 60 minutes, just after stopping the pumping. Determine the yield from the well of 2m diameter when the depression head in 3 m.* (10)

[**Hint.** Follow exactly similar Solved Example 16.11 and compute :

$$\frac{C'}{A} = \frac{2.3}{3600}\log_{10}\frac{2.5}{1.5} = 1.417 \times 10^{-4}$$

Q = yield from a well of 2m dia with 3m depression head

$$= \left(\frac{C'}{A}\right) \cdot A \cdot s = (1.417 \times 10^{-4}) \times \left(\frac{\pi}{4} \times 2^2\right) \times 3\ \text{m}^3/\text{s}$$

$= \mathbf{1.335}\ l/s$ **Ans.**]

Q. 8. (*b*) *Explain the purpose of providing groins. State the factors which affect the selection of type and performance of groin. Mention about the orientation, desirable length and spacing of groins in a groin system.* (10)

[Please refer article 8.5.3 (3)]

Year 2002 (*Engineering Services*)

Q. 2. (*b*) *For a drainage basin of 640 km², isohyetals based on a storm event yield the following data :*

Isohyetal interval (cm)	*14-12*	*12-10*	*10-8*	*8-6*	*6-4*	*4-2*	*2-0*
Inter isohyetal area (km²)	*90*	*140*	*125*	*140*	*85*	*40*	*20*

Estimate the average depth of precipitation over the basin.

[**Hint :** Please follow exactly similar solved example 7.5, and compute]

$$P = \frac{90 \times 13 + 140 \times 11 + 125 \times 9 + 140 \times 7 + 85 \times 5 + 40 \times 3 + 20 \times 1}{90 + 140 + 125 + 140 + 85 + 40 + 20 = 640}$$

$$= \mathbf{8.41\ cm} \quad \textbf{Ans.}]$$

Q5. (*b*) *Design a regime channel for a discharge of 35 m³/s with silt factor of 0.9 by Lacey's theory, taking side slopes as 1 H : 2 V.*

[**Hint :** Please follow exactly similar solved Question 4.9 ; and compute

$B = 24$ m, $y = 1.83$ m, $S_o = 1$ in 7200 **Ans.**]

Q7. (*a*) *At an energy dissipater structure below a low spillway, the discharge is 19 m²/s, and the energy loss is 1 m at the hydraulic jump forming therein. Determine the depth of flow at both ends of the jump.*

Solution. $q = 19\ \text{m}^2/s$; $H_L = 1$ m ; $y_1 = ?$; $y_2 = ?$

Use equations as :

$$y_1 y_2 (y_2 + y_1) = \frac{2q^2}{g} \qquad \text{...(Eqn. 10.5)}$$

and

$$H_L = \frac{(y_2 - y_1)^3}{4 y_1 y_2} \qquad \text{...(Eqn. 10.12)}$$

Solve the above equations for two unknowns, y_1 and y_2, as follows :

From Eqn. (10.5), we get

$$y_1 y_2 (y_2 + y_1) = \frac{2 \times 19^2}{9.81} = 73.6 \qquad \text{...}(i)$$

From Eqn. (10.12), we get

$$1 = \frac{(y_2 - y_1)^3}{4 \cdot y_1 y_2} \qquad \text{...}(ii)$$

Let $$\frac{y_2}{y_1} = x$$

$\therefore$ $$y_2 = y_1 \cdot x$$

Substituting in (*i*), we get

$$y_1 \cdot y_1 x (y_1 x + y_1) = 73.6$$

or $$y_1^2 \cdot x \cdot y_1 (x + 1) = 73.6$$

or $$x \cdot y_1^3 (x + 1) = 73.6 \qquad \text{...}(iii)$$

Also from (*ii*),

$$1 = \frac{(y_1 x - y_1)^3}{4 \cdot y_1 \cdot x \cdot y_1} = \frac{y_1^3 (x-1)^3}{4 \cdot x \cdot y_1^2} = \frac{y_1}{4x} (x-1)^3$$

or $$4x = y_1 (x-1)^3$$

or $$y_1 = \frac{4x}{(x-1)^3} \quad ...(iv)$$

Substituting in (*iii*), we get

$$x \cdot \left[\frac{4x}{(x-1)^3}\right]^3 [x+1] = 73.6$$

Solve by hit and trial, for x, as :

$$x = \mathbf{2.548}$$

$$\therefore \quad y_1 = \frac{4 \times 2.548}{(2.548-1)^3} = \mathbf{2.748\ m}$$

$$y_2 = y_1\, x = 2.748 \times 2.548 = \mathbf{7.0\ m}$$

Hence, the depth at both ends, of jump, y_1 and y_2 will be 2.748 m and 7.0 m, respectively. Ans.

Q8. (*a*) *The daily flows in a river for three consecutive years are given in the table by class interval along with the number of days the flow belonged to this class. What are the 50% & 75% dependable flows for the river ?*

Daily mean discharge m^3/s		*100-90.1*	*90-80.1*	*80-70.1*	*70-60.1*	*60.50.1*
No. of days in each class interval	1981	0	16	27	21	43
	1982	6	19	25	60	51
	1983	10	16	38	67	58

Daily mean discharge m^3/s		*50-40.1*	*40-30.1*	*30-20.1*	*20-10.1*	*10-Negligible*
No. of days in each class interval	1981	59	64	22	59	54
	1982	38	29	48	63	26
	1983	38	70	29	26	13

[Hint : Please refer Solved Example 18.3]

Year 2003 (Engineering Services)

Q2. (*b*) *During the passage of a flood the following data was estimated at two sections 500 m apart :*

Sections	*Water surface elevation, m*	*Area of flow section, m^2,*	*Hydraulic mean depth, m*
Upstream, *P*	85.233	91.746	2.835
Downstream, *R*	85.176	84.354	2.917

The eddy loss coefficient for gradual contraction is to be taken as 0.1 and for gradual expansion as 0.35. Estimate the flood discharge passing through the reach. $n = 0.022$. (15)

[**Hint.** Follow exactly similar solved example 7.23]

Q5. (*b*) *Water discharges at the rate of 298 m^3/sec over a spillway 14 m wide into a stiling basin of the same width. The water level behind the spillway is 61 m over datum. The river water level in the downstream is 29.2 m over datum. The river bed is paved flush with the bed level to just adequate length to form the jump basin. Assuming that no energy is dissipated in the flow down the spillway, find the basin invert level required for a hydraulic jump to be formed within the basin* (15)

[**Solution.** Here $H_L = (61.0 - 29.2)$ m = 31.8 m

$$q = \frac{Q}{B} = \frac{298}{14} = 21.29 \text{ m}^3/\text{s/m}$$

This is a case where H_L and q are known, and we have to find the value of y_2, since the invert level of jump basin will be equal to (RL 29.2 – y_2). To determine y_2, proceed as given in article 10.3.1—when Blench curves are not available.

From

$$\text{Determine } y_c = \sqrt[3]{\frac{q^2}{g}} \text{ (i.e. Eq. 10.3)} = \sqrt[3]{\frac{(21.29)^2}{9.81}} = 3.59 \text{ m}$$

Now use Eq. (10.6) as :

$$Z = \frac{H_L}{y_c} = \frac{31.8}{3.59} = 8.86$$

Now use Eq. (10.10) as :

$$Y = 1 + 0.93556 \, . \, Z^{0.24} \text{ for } Z > 1$$

For $Z = 8.86$ (> 1) $\quad Y = 1 + 0.93556 \times (8.86)^{0.24} = 2.58$

Now, use Eq. (10.5) as :

$$\frac{y_2}{y_c} = Y$$

or $$\frac{y_2}{3.59} = 2.58 \qquad \text{or} \quad y_2 = 9.26 \text{ m}$$

The basin invert level = 29.2 – 9.26 = **19.94 m** above datum. **Ans.**]

Q6. (*b*) *A channel is 40 m wide, 2.6 m deep with slope of 1 in 5000. The representative bed material size D_{65} is 0.64 mm, and mean diameter of the bed material is 0.38 mm. Mannings $n = 0.021$. Neglecting the side friction, find the bed load transported by the channel in tonnes per day by Meyer-Peter equation, considering B/D as large and $\tau_c = 0.075$ d.*

Solution. Meyer-Peter's equation, giving bed load transport in N/m.s is given by Eq. (4.51) as :

$$g_b = 0.417\left[\tau_0\left(\frac{n'}{n}\right)^{3/2} - \tau_c\right]^{3/2}$$

where $\tau_0 = 0.97\ \gamma_w$ R.S. for the channel

where γ_w = unit wt. of water

$= 9.81\ \text{kN/m}^3 = 9.81 \times 10^3\ \text{N/m}^3$

$R = \frac{A}{P} = \frac{B \cdot y}{B} = y$ for wider channels having B/y large

$= 2.6$ m

$$S = \frac{1}{5000}$$

$$\therefore \quad \tau_0 = 0.97 \times (9.81 \times 1000) \times 2.6 \times \frac{1}{5000} = \mathbf{4.948\ N/m^2}$$

$n' = 1/24 \,.\, d^{1/6}$ where d is the representative size of the bed material in m, usually taken as median size (d_{50}) or d_{65}, which is given as equal to 0.64 mm.

$$\therefore \quad d = 0.64 \times 10^{-3}$$

$$\therefore \quad n' = \frac{1}{24}(0.64 \times 10^{-3})^{1/6} = \mathbf{0.01223}$$

$\boldsymbol{n = 0.021}$ **(given)**

τ_c is usually given as $\tau_c = 0.687 \,.\, d_a$ when τ_c is in N/m^2, and d_a is in mm.

Here it is given that $\tau_c = 0.075d$. Eventually, the given eqn. is meant for τ_c, when τ_c is expressed in kg/m^2. For S.I. units, the given eqn. will hence become.

$\tau_c = 0.075\, d_a \times 9.81\ \text{N/m}^2$

$= 0.736\, d_a\ (\text{N/m}^2)$

d_a is mean size of the bed particles, which is given as equal to 0.38 mm.

$$\therefore \quad \tau_c = 0.736 \times 0.38\ \text{N/m}^2 = \mathbf{0.279\ N/m^2}$$

Now, $$g_b = 0.417\left[4.948\left(\frac{0.01223}{0.021}\right)^{3/2} - 0.279\right]^{3/2}$$

$$= 0.417\,[2.199 - 0.279]^{3/2} = 0.417\,(1.92)^{1.5}$$

or $$g_b = 1.109\ \text{N/m.s} = \frac{1.109}{9.81}\ \text{kg/s/m}$$

∴ Bed load transported by 40 m wide channel

$$= \frac{1.109}{9.81} \times 40\ \text{kg/sec} = 4.522\ \text{kg/sec}$$

$$= \frac{4.522 \times 24 \times 3600}{1000}\ \text{t/day} = \textbf{390.69 tonnes/day. Ans}]$$

Q7. (*a*) *Flood frequency computations for a flashy river at a point 50 km upstream of a bund site indicated the following :*

Return period, T_{year}	*50*	*100*
Peak flood, m^3/sec	*20,600*	*22,150*

Estimate the flood magnitude in the river with a return period of 500 year through use of Gumbel's method (15)

[**Hint.** Follow exactly similar Solved Example 7.58.]

Q8. (*a*) *Given below are the monthly rainfall, P, and the corresponding run off values, R, for a period of 10 months for a catchment. Develops a correlation between R and P :*

Month No.	*P, cm*	*R, cm*
1	*4*	*0.2*
2	*22*	*7.1*
3	*28*	*10.9*
4	*15*	*4.0*
5	*12*	*3.0*
6	*8*	*1.3*
7	*4*	*0.4*
8	*15*	*4.1*
9	*10*	*2.0*
10	*5*	*0.3*

[**Hint.** To obtain a linear relation between rainfall P and runoff Q (here represented by R), we have to plot a curve taking R on X-axis and P on Y-axis, as to plot a curve like the one obtained in Fig. 7.105. The slope of straight line portion is then determined to compute the desired relation between P and R. This relation is worked out as

$R = 0.47P$ in Fig. 3.11. **Ans.**

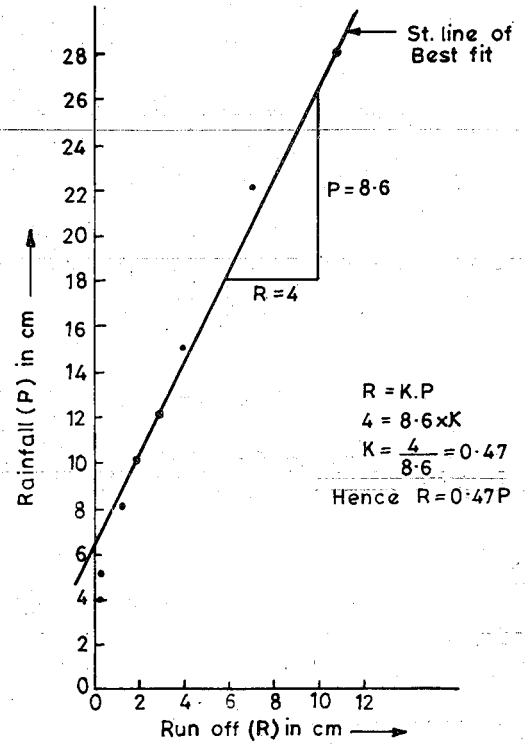


Fig. 31.11

Year 2004 (*Engineering Services*)

Q3. (*a*) *A watershed has five non-recording rain gauges, installed in its area. The amount of rainfall recorded for one of the years is given below :*

Rain gauge station	I	II	III	IV	V
Annual rainfall in cm	100	120	190	95	125

Find the required optimum number of non-recording and recording rain gauges for this watershed. Assume an error of 10% in the estimation of mean rainfall. (15)

[**Hint.** Pl. follow exactly similar solved example 7.3]

Q6. (*a*) *Explain briefly the functions of the following for a typical canal headworks :*

(*i*) *divide wall,*

(*ii*) *guide bank,*

(*iii*) *sediment ejector.*

Explain briefly how does seepage endanger the safety of a hydraulic structure on permeable foundation. (15)

Q7. (*a*) *In a catchment, the average rainfall for a storm at two successive 6 hr intervals was 3.0 cm and 6.0 cm respectively. The abstraction losses ϕ-index, were estimated to be 0.20 cm/hour. For the same catchment, the calculated data for a 6 hour unit hydrograph is available and is given below. Find the direct run-off hydrograph due to the storm.*

Time (hours)	0	6	12	18	24	30	36	42	48	54	60
Unit hydrograph ordinate (m^3/s)	0	10	25	40	100	150	100	75	25	15	0

(20)

[Hint. Pl. follow similar solved question 7.35. The excess rainfall here, in two successive 6 hr rainfall periods, will be [3.0 – 6 × 0.2 = 1.8 cm] and [6 – 6 × 0.2 = 4.8 cm], respectively. The ordinates of D.R.H. for first rain will be calculated by multiplying the given ordinates of U.H. by 1.8. While the ordinates of DRH caused by 2nd rain will be obtained by multiplying the given ordinates of U.H. by 4.8. The ordinates due to 2nd rain will start after 6 hr from those of the first, as usual. The summation of the two set of ordinates will give ordinates of total S.R. hydrograph.]

Q8. *(a) Define, duty, delta and base period of a crop and express the relationship connecting them. Explain briefly with the help of a diagram the concept of 'frequency of irrigation'.* (10)

(b) A loam soil has a field capacity of 25 per cent and wilting coefficient of 10 per cent. The dry unit weight of soil is 1.5 g/cc. If the root zone depth is 60 cm, determine the storage capacity of the soil. Irrigation water is applied when moisture content falls to 15 per cent. If the water application efficiency is 75 per cent, determine the water depth required to be applied in the field. (10)

[Solution. Pl. follow exactly similar solved question 2.15 and compute as :

Max. storage capacity in root zone of depth, d

$$= \frac{\gamma_d \cdot d}{\gamma_w}\left[\frac{\text{Field capacity mc}}{100} - \frac{\text{Wilting pt. mc}}{100}\right]$$

$$= \frac{\rho_d \cdot d}{\rho_w}\left[\frac{\text{F.C.}}{100} - \frac{\text{W.P.}}{100}\right]$$

$$= \frac{1.5\,\frac{\text{gm}}{\text{cc}} \times 60\text{ cm}}{1\,\frac{\text{gm}}{\text{cc}}}\left[\frac{25}{100} - \frac{10}{100}\right]$$

$$= 90\left[0.25 - 0.10\right]\text{ cm} = \mathbf{13.5\ cm.}\quad \textbf{Ans.}$$

Irrigation water is to be supplied when *mc* falls to 15%. It means that m.c. is allowed to vary between 25% (F.C. mc) and 15%. The deficiency of water depth created during this fall of m.c.

$$= \frac{\rho_d \cdot d}{\rho_w}[0.25 - 0.15]$$

$$= 1.5 \times 60\text{ cm} \times 0.10 = \mathbf{9\ cm.}$$

Hence, 9 cm depth of water is the net irrigation requirement.

Water depth required in the field

$$= \frac{\text{Net irrigation requirement}}{\text{Water application efficiency}}$$

$$= \frac{9\text{ cm}}{0.75} = \mathbf{12\ cm.}\quad \textbf{Ans.}$$

CIVIL SERVICES COMPETITION (IAS)

This competition is open to all graduates including Engineering graduates. The age limits vary between 21 to 30 years as on 1st August of the year of exam.

In this competition, you would have to, first of all, appear in a screening test, called the **Preliminary Examination.** This exam. will consists of *objective type questions*, having the following marks :

(*i*) *General Studies*	:	150 Marks
(*ii*) *One subject, to be selected from out of 23 optionals, listed below, such as Civil Engineering (for you)*	:	300 Marks
	Total	450 Marks

List of optionals for Prelimes :

(*i*) *Agriculture;* (*ii*) *Animal Husbandry & Veterinary Science;* (*iii*) *Botany;* (*iv*) *Chemistry;* (*v*) *Civil Engineering;* (*vi*) *Commerce;* (*vii*) *Economics;* (*viii*) *Electrical Engg;* (*ix*) *Geography;* (*x*) *Geology;* (*xi*) *Indian History;* (*xii*) *Law;* (*xiii*) *Mathematics;* (*xiv*) *Mechanical Engg;* (*xv*) *Medical Science;* (*xvi*) *Philosophy;* (*xvii*) *Physics;* (*xviii*) *Political Science;* (*xix*) *Psychology;* (*xx*) *Public Administration;* (*xxi*) *Sociology;* (*xxii*) *Statistics; and* (*xxiii*) *Zoology.*

The syllabus (**revised wef year 2000**) prescribed for "General Studies" and "Civil Engineering" objective type papers, is given below :

Preliminary Exam
Paper I — General Studies (Max. Marks 150)

The paper on General Studies will include Questions covering the following field of knowledge

General Science.

Current events of national and international importance.

History of India and Indian National Movement.

Indian and World Geography

Indian Polity and Economy.

General Mental Ability

Questions on General Science will cover General appreciation and understanding of science including matters of everyday observation and experience, as may be expected of a well educated person who has not made a special study of any particular scientific discipline. In current events, knowledge of significant national and international events will be tested. In History of India, emphasis will be on broad general understanding of the subject in its social, economic and political aspects. Questions on the Indian National Movement will relate to the nature and character of the nineteenth century resurgence, growth of nationalism and attainment of Independence. In Geography, emphasis will be on Geography of India. Questions on the Geography of India will relate to physical, social and economic Geography of the country, including the main features of Indian agricultural and natural resources. Questions on Indian Polity and Economy will test knowledge of the country's political system and Constitution of India, Panchayati Raj, Social Systems and economic developments in India. On general mental ability, the candidates will be tested on reasoning and analytical abilities.

Paper II — Civil Engineering (Max. Marks 300)
PART-A

1. **Engineering Mechanics.** Units and dimensions, SI Units, Vectors, Concept of

Force, Concept of particle and rigid body. Concurrent, nonconcurrent and parallel forces in a plane, moment of force and Varignon's theorem, free body diagram, conditions of equilibrium, Principle of virtual work, equivalent force system.

First and Second Moment of area, Mass moment of Inertia.

Static Friction Inclined plane and bearings.

Kinematics and Kinetics : Kinematics in cartesian and polar co-ordinates, motion under uniform and non-uniform acceleration, motion under gravity. Kinetics of particle : Momentum and Energy principles, D Alembert's Principle, Collision of elastic bodies, rotation of rigid bodies, simple harmonic motion.

2. **Strength of Materials.** Simple Stress and Strain, Elastic constants, axially loaded compression members, Shear force and bending moment, theory of simple bending, Shear Stress distribution across cross-sections, Beams of uniform strength, Leaf spring, Strain Energy in direct stress, bending and shear.

Deflection of beams. Macaulay's method, Mohr's moment area method, Conjugate beam method, unit load method. Torsion of Shafts, Transmission of power, closecoiled helical springs, Elastic stability of columns : Euler's, Rankine's and Secant formulae. Principal Stresses and Strains in two dimension, Mohr's Circle. Theories of Elastic Failure, Thin and Thick cylinders : Stresses due to internal and external pressures-Lame's equation.

3. **Structural Analysis.** Analysis of pin jointed plane trusses, deflection in trusses. Three hinged and two hinged arches, rib shortening, temperature effects, influence lines in arches. Analysis of propped cantilevers, fixed beams, continuous beams and rigid frames. Slope deflection, moment distribution, Kani's method and Matrix method : Force and Displacement methods. Rolling loads and influence lines for determinate beams and pin jointed trusses.

PART-B

4. **Geotechnical Engineering.** Types of soil, field identification and classification, phase relationships, consistency limits, particle size distribution, classification of soil, structure and clay mineralogy.

Capillary water and structural water, effective stress and pore water pressure, Darcy's Law, factors affecting permeability, determination of permeability, permeability of stratified soil deposits.

Seepage pressure, quick sand condition, compressibility and consolidation, Terzaghi's theory of one dimensional consolidation, consolidation test. Compaction of soil, optimum moisture content, proctor Density.

Subsurface exploration, methods of boring, sampling, types of sampler, filed tests.

Shear strength of soils, Mohr-Coulomb failure theory, shear tests, Earth pressure at rest, active and passive pressures, Rankine's theory, Coulomb's wedge theory, earth pressure on retaining wall.

Bearing capacity, Terzaghi and other important theories, net and gross bearing pressure, Immediate and consolidation settlement.

Load carrying capacity of pile groups.

Stability of slope-Conventional method of slices, stability numbers.

5. **Transportation Engineering.** Highway alignment, choice of layout and capacity of highways, location survey, geometric design of highways-various elements, curves, grade separation and segregation of traffic, Intersection design, highway materials and testing subgrade and pavement components, types of pavements, road drainage, elements of airport engineering.

Railway engineering-elements of permanent track-rails, sleepers, ballast and rail fastenings, tractive resistance, elements of geometric design-gradients and grade compensation on curves, cant transition curves and vertical curves, stresses in railway tracks, points and crossings, signalling and interlocking, maintenance of railway track. Culverts and small bridges.

PART-C

6. **Fluid Mechanics.** Fluid properties, fluid statics, forces on plane and curved surfaces, stability of floating and submerged bodies.

Kinematics. Velocity, streamlines, continuity equation, accelerations irrotational and rotational flow, velocity potential and stream functions, flownet, separation.

Dynamics. Euler's equation along streamline, control volume equation, continuity, momentum, energy and moment of momentum equation from control volume equation, applications to pipe flow, moving vanes, moment of momentum, Dimensional analysis. Boundary layer on a flat plate, drag and lift on bodies. Laminar and Turbulent Flows. Laminar and turbulent flow through pipes, friction factor variation, pipe networks, water hammer, and surge tanks.

Open Channel Flow. Energy and momentum correction factors, uniform and non-uniform flows, specific energy and specific force, critical depth, Friction factors and roughness coefficients, flow in transitions, free overfall, weirs, hydraulic jump, surges, gradually varied flow equations, surface profiles, moving hydraulic jump.

PART-D

7. **Environmental Engineering**

Water supply. Estimation of surface and subsurface water resources, predicting demand for water, impurities of water and their significance, physical, chemical and bacteriological analysis, water borne diseases, standards for potable water.

Intake of water. pumping and gravity schemes, water treatment : principles of coagulation, floculation and sedimentation; slow-, rapid-, pressure-, filters; chlorination, softening, removal of taste, odour and salinity.

Water storage and distribution : storage and balancing reservoir types, location and capacity. Distribution systems : layout, hydraulics of pipe lines, pipe fittings, valves including check and pressure reducing valves, meters, analysis of distribution systems, leak detection, maintenance of distribution systems, pumping stations and their operations.

Sewerage systems. Domestic and industrial wastes, storm sewage-separate and combined systems, flow through sewers, design of sewers, sewer appurtenances, manholes, inlets, juctions, siphon. Plumbing in Public buildings.

Sewage characterisation. BOD, COD, solids, dissolved oxygen, nitrogen and TOC. Standards of disposal in normal water course and on land.

Sewage treatment. Working principles, units, chambers, sedimentation tank, trickling filters, oxidation ponds, activated sludge process, septic tank, disposal of sludge, recycling of waste water.

9. **Construction Management.** Elements and principles of Activity on Arrow (AOA) and Activity on Node (AON) networks and work breakdown structure. Interfaces. Ladder networks. Activity time. Time computations and floats. ATC and PTC trade-off. Work study and sampling. Scheduling principles-material schedules. ABC and EOQ analysis of inventory. Budgeting with barcharts. Working capital, PERT, probability of completion.

Elements of Engineering Economics, methods of appraisal, present worth, annual-cost, benefit-cost, incremental analysis. Economy of scale and size. Choosing between alternatives including levels of investments, Project profitability.

This preliminary examination is meant to serve as a screeining test only. The marks

obtained in the preliminary Exam by the candidates who are declared qualified for admission to Main Examination will not be counted for determining their final order of merit. The number of candidates to be admitted to the Main Examination will be about twelve to thirteen times the total approximate number of vacancies to be filled in various Services & Posts. Only those candidates who are declared by the commission to have qualified in the Preliminary Examination in a year will be eligible for admission to the Main Examination of that year, provided they are otherwise eligible for admission to the Main Examination.

Main Examination

The main examination consists of one qualifying paper in *English* and another qualifying paper in *one of the Indian Languages* to be selected from the languages included in the eighth schedule of constitution, such as Hindi, Tamil, Bangali, Sanskrit or any other regional language. These two papers, each carrying 300 marks, will be of matriculation standard, and of qualifying in nature *i.e.* the *marks obtained in these papers will not be counted in your final ranking in the competition.*

Besides the above two qualifying papers (Paper I and II), there will be 7 other papers (Paper III to IX), full marks of which will count in your final ranking at this competition. These 7 papers will consists of ;

Compulsory Subjects		
Paper III	Essay	200 marks
Paper IV	General Studies I	300 marks
Paper V	General Studies II	300 marks
Paper VI	Paper I of chosen Optional I	300 marks
Paper VII	Paper II of chosen Optional I	300 marks
Paper VIII	Paper I of chosen Optional II	300 marks
Paper IX	Paper II of chosen Optional II	300 marks
		2000 marks
	Interview and Personality Test	300 marks
	Grand Total	**2300 marks**

Note : *On the basis of merit list drawn out of 2000 marks of the Main Examination, cendidates will be summoned for Interview. The no. of condidates called for Interview will be about Twice the number of vacancies to be filled. The final select list will be drawn on the basis of merit for total marks obtained out of 2300 marks.*

Obviously, all the civil engineering students are expected to offer *'Civil Engineering'* subject as one of their two chosen Optionals. Hence, the subject will cover 600 marks out of the grand total of 2300 marks at this competition. You may offer 2nd optional as either "Geology" or "Mathematics" or any other subject of your interest.

The syllabuses (**revised wef year 2000**) for both the papers on *'General Studies'* as well as on the subject of *'Civil Engineering'* are given below for your ready reference.

A perusal of the syllabus of 'Civil Engineering' subject further reflects that **Paper I** will cover four subjects like *Theory of Structures, Structural Designs, Fluid Mechanics* and *Soil Mechanics.* **Paper II** will cover the subjects of *Construction Technology, Survey & Transportation, Hydrology, Water Resources, Irrigation,* and *Environmental Engineering.* Both the papers will thus cover virtually the entire field of civil engineering.

ESSAY

Candidates will be required to write an essay on a specific topic. The choice of subjects will be given. They will be expected to keep closely to the subject of the essay to arrange their ideas in orderly fashion, and to write concisely. Credit will be given for effective and exact expression.

GENERAL STUDIES (MAIN EXAM.)

Note. *The nature and standard of questions in the General studies papers will be such that a well-educated person will be able to answer them without any specialized study. The questions will be such as to test a candidate's general awareness of a variety of subjects, which will have relevance for a career in Civil Services.*

Syllabus for General Studies Paper I (Main Exam)

(*a*) *History of Modern India and Indian Culture.* The History of Modern India will cover history of the Country from about the middle of nineteenth century and would also include questions on important personalities who shaped the Freedom Movement and Social reforms. The part relating to Indian Culture will cover all aspects of Indian Culture from the ancient to modern times.

(*b*) *Geography of India.* In this part, questions will be on the physical, economic and social geography of India.

(*c*) *Indian Polity.* This part will include questions on the Constitution of India, Political system and related matters.

(*d*) *Current National issues and topics of social relevance.* This part is intended to test the Candidate's awareness of current national issues and topics of social relevance in the present-day India, such as the following.

Demography & Human Resource & related issues. Behavioural & Social issues & Social Welfare problems, such as child labour, gender equality, adult literacy, rehabilitation of the handicapped and other deprived segments of the society, drug abuse, public health, etc.

Law enforcement issues, human rights, corruption in public life, communal harmony etc. Internal Security and related issues.

Environmental issues, ecological preservation, conservation of natural resources and national heritage.

The role of national institutions, their relevance and need for change.

Syllabus for General Studies Paper II (Main Exam)

(*a*) *India and the World.* This part is intended to test candidate's awareness of India's relationship with the world in various spheres, such as the following :-

(*i*) Foreign Affairs (*ii*) External Security and related matters

(*iii*) Nuclear Policy (*iv*) Indians abroad

(*b*) *Indian Economy.* In this part, questions will be on the planning and economic development in India, economic & trade issues, Foreign Trade, the role and functions of I.M.F., World Bank, W.T.O. etc.

(*c*) *International Affairs & Institutions.* This part will include questions on important events in world affairs and on international institutions.

(*d*) *Developments in the field of science & technology, Communications and space.* In

this part, questions will test the candidate's awareness of the developments in the field of science & technology, communications and space and also basic ideas of computers.

(*e*) *Statistical analysis, graphs and diagrams.* This part will include exercises to test the candidate's ability to draw common sense conclusions from information presented in statistical, graphical or diagrammatical form and to point out deficiencies, limitations or inconsistencies therein.

OPTIONAL SUBJECTS FOR MAIN EXAMINATION

Total number of questions in the question papers of optional subjects will be eight. All questions will carry equal marks. Each paper will be divided into two parts, viz. Part A and Part B, each part containing four questions. Out of eight questions, five questions are to be attempted. One question in each part will be compulsory. Candidates will be required to answer three more questions out of the remaining six questions, taking at least one question from each part. In this way, at least two questions will be attempted from each Part *i.e.* one compulsory question plus one more.

You have to choose 2 optionals for the Main Exam, out of the following subjects :

Optionals for Main Exam (2 optionals to be chosen from 50 subjects)

(*i*) Agriculture
(*ii*) Animal Husbandry and Veterinary Science
(*iii*) Anthropology
(*iv*) Botony
(*v*) Chemistry
(*vi*) Civil Engineering
(*vii*) Commerce and Accountancy
(*viii*) Economics
(*ix*) Electrical Engineering
(*x*) Geography
(*xi*) Geology
(*xii*) History
(*xiii*) Low
(*xiv*) Arabic
(*xv*) Bengali
(*xvi*) Chinese
(*xvii*) English
(*xviii*) French
(*xix*) German
(*xx*) Gujarati
(*xxi*) Hindi
(*xxii*) Kannada
(*xxiii*) Kashmiri
(*xxiv*) Konkani
(*xxv*) Malayalam
(*xxvi*) Manipuri
(*xxvii*) Marathi
(*xxviii*) Nepali
(*xxix*) Oriya
(*xxx*) Pali
(*xxxi*) Persian
(*xxxii*) Punjabi
(*xxxiii*) Russian
(*xxxiv*) Sanskrit
(*xxxv*) Sindhi
(*xxxvi*) Tamil
(*xxxvii*) Telgu
(*xxxviii*) Urdu
(*xxxix*) Management
(*xxxx*) Mathematics
(*xxxxi*) Mechanical Engineering
(*xxxxii*) Medical Science
(*xxxxiii*) Philosophy
(*xxxxiv*) Physics
(*xxxxv*) Political Science and International Relations
(*xxxxvi*) Psychology
(*xxxxvii*) Public Administration
(*xxxxviii*) Sociology
(*xxxxix*) Statistics
(*xxxxx*) Zoology

Since all the civil Engg. students are expected to offer "Civil Engineering" as one of their two optionals, we are giving below the syllabus of this subject :

Syllabus (*Revised wef year 2000*) for Civil Engineering Paper I (Main Exam.)

Part A — *Engineering Mechanics, Strength of Materials and Structural Analysis*

1. **Engineering Mechanics.** Units and Dimensions, SI Units, Vector, Concept of Force, Concept of particle and rigid body. Concurrent, Non Concurrent and parallel forces in a plane, moment of force and Varignon's theorem, free body diagram, conditions of equilibrium, Principle of virtual work, equivalent force system.

First and Second Moment of area, Mass moment of Inertia.

Static Friction, Inclined Plane and bearings. Kinematics and Kinetics :

Kinematics in Cartesian and Polar Co-ordinates, motion under uniform and non-uniform acceleration, motion under gravity. Kinetics of particle : Momentum and Energy principles, D' Alembert's Principle, Collision of elastic bodies, rotation of rigid bodies, simple harmonic motion, Flywheel.

2. **Strength of Materials.** Simple Stress and Strain, Elastic constants, axially loaded compression members, Shear force and bending moment, theory of simple bending, Shear Stress distribution across cross-sections, Beams of uniform strength, Leaf spring. Strain Energy in direct stress, bending & shear.

Deflection of beams : Meaculay's method, Mohr's Moment area method, Conjugate beam method, unit load method. Torsion of Shafts, Transmission of power, close coiled helical springs, Elastic stability of columns, Euler's, Rankine's and Secant formulae. Principal Stresses and Strains in two dimensions. Mohr's Circle, Theories of Elastic Failure, Thin and Thick cylinder : Stresses due to internal and external pressure - Lame's equations.

3. **Structural Analysis.** Castiglianio's theorems I and II, unit load method of consistent deformation applied to beams and pin jointed trusses. Slope deflection, moment distribution, Kani's method of analysis and Column Analogy method applied to indeterminate beams and rigid frames.

Rolling loads and Influence lines : Influence lines for Shear Force and Bending moment at a section of beam. Criteria for maximum shear force and bending Moment in beams traversed by a system of moving loads. Influence lines for simply supported plane pin jointed trusses.

Arches : Three hinged, two hinged and fixed arches, rib shortening and temperature effects, influence lines in arches.

Matrix methods of analysis : Force method and displacement method of analysis of indeterminate beams and rigid frames.

Plastic Analysis of beams and frames : Theory of plastic bending, plastic analysis, statical method, Mechanism method.

Unsymmetrical bending : Moment of inertia, product of inertia, position of Neutral Axis and Principle axes, calculation of bending stresses.

Part B — *Design of structures. Steel, Concrete and Masonry Structures*

1. **Structural Steel Design.** Structural Steel : Factors of safety and load factors. Rivetted, bolted and welded joints and connections. Design of tension and compression members, beams of built up section, rivetted and welded plate girders, gantry girders, stancheons with battens and lacings, slab and gussetted column bases. Design of highway and railway bridges : Through and deck type plate girder. Warren girder, Pratt truss.

2. **Design of Concrete and Masonry Structures.** Concept of mix design. Reinforced Concrete : Working Stress and Limit State method of design-Recommendations

of I.S. codes. Design of one way and two way slabs, stair-case slabs, simple and continuous beams of rectangular, T and L sections. Compression members under direct load with or without eccentricity, Isolated and combined footings.

Cantilever and Counterfort type retaining walls.

Water tanks : Design requirements for Rectangular and circular tanks resting on ground.

Prestressed concrete : Methods and systems of prestressing, anchorages, Analysis and design of sections for flexure based on working stress, loss of prestress.

Design of brick masonry as per I.S. Codes. Design of masonry retaining walls.

Part C. *Fluid Mechanics, Open Channel Flow and Hydraulic Machines*

1. **Fluid Mechanics.** Fluid properties and their role in fluid motion, fluid statics including forces acting on plane and curve surfaces. Kinematics and Dynamics of Fluid flow : Velocity and accelerations, streamlines, equation of continuity, irrotational and rotational flow, velocity potential and stream functions, flownet, methods of drawing flownet, sources and sinks, flow separation, free and forced vertices.

Control volume equation, continuity, momentum, energy and moment of momentum equations from control volume equation, Navier-Stokes equation, Euler's equation of motion, application to fluid flow problems, pipe flow, plane, curved, stationary and moving vanes, sluice gates, weirs, orifice meters and Venturi meters.

Dimensional Analysis and Similitude. Buckingham's Pi- theorem, dimensionless parameters, similitude theory, model laws, undistorted and distorted models.

Laminar Flow. Laminar flow between parallel, stationary and moving plates, flow through tube.

Boundary layer. Laminar and turbulent boundary layer on a flat plate, laminar sublayer, smooth and rough boundaries, drag and lift.

Turbulent flow through pipes : Characteristics of turbulent flow, velocity distribution and variation of pipe friction factor, hydraulic grade line and total energy line, siphons, expansion and contractions in pipes, pipe networks, water hammer in pipes and surge tanks.

2. **Open Channel Flow.** Uniform and non-uniform flows, momentum and energy correction factors, specific energy and specific force, critical depth, resistance equations and variation of roughness coefficient, rapidly varied flow, flow in contractions, flow at sudden drop, hydraulic jump and its applications, surges and waves, gradually varied flow, classification of surface profiles, control section, step method of integration of varied flow equation, moving surges and hydraulic bore.

3. **Hydraulic Machines and Hydropower.** Centrifugal pumps-Types, characteristics, Net Positive Suction Height (NPSH), specific speed. Pumps in parallel.

Reciprocating pumps, Air vessels, Hydraulic ram, efficiency parameters, Rotary and positive displacement pumps, diaphragm and jet pumps.

Hydraulic turbines, types classification, Choice of turbines, performance parameters, controls, characteristics, specific speed.

Principles of hydropower development. Type, layouts and Component works. Surge tanks, types and choice. Flow duration curves and dependable flow. Storage and pondage. Pumped storage plants. Special features of mini micro-hydel plants.

Part D — *Geo-technical Engineering*

Types of soil, phase relationships, consistency limits, particles size distribution, classifications of soil, structure and clay mineralogy.

Capillary water and structural water, effective stress and pore water pressure, Darcy's Law, factors affecting permeability, determination of permeability, permeability of stratified soil deposits.

Seepage pressure, quick sand condition, compressibility and consolidation, Terzaghi's theory of one dimensional consolidation, consolidation test.

Compaction of soil, field control of compaction. Total stress and effective stress parameters, pore pressure coefficients.

Shear strength of soils, Mohr Coulomb failure theory, Shear tests.

Earth pressure at rest, active and passive pressures, Rankine's theory, Coulomb's wedge theory, earth pressure on retaining wall, sheet pile walls, Braced excavation.

Bearing capacity, Terzaghi and other important theories, net and gross bearing pressure.

Immediate and consolidation settlement. Stability of slope, Total Stress and Effective Stress methods, Conventional methods of slices, stability number.

Subsurface exploration, methods of boring, sampling, penetration tests, pressure meter tests.

Essential features of foundation, types of foundation, design criteria, choice of type of foundation, stress distribution in soils, Boussinessq's theory, Newmarks's chart, pressure bulb, contact pressure, applicability of different bearing capacity theories, evaluation of bearing capacity from field tests, allowable bearing capacity, Settlement analysis, allowable settlement.

Proportioning of footing, isolated and combined footings, rafts, buoyancy rafts, Pile foundation, types of piles, pile capacity, static and dynamic analysis, design of pile groups, pile load test, settlement of piles, lateral capacity. Foundation for Bridges. Ground improvement techniques-preloading, sand drains, stone column, grouting, soil stabilisation.

Syllabus (*Revised wef year 2000*) for Civil Engineering Paper II (Main Exam.)

Part A — *Construction Technology, Equipment, Planning and Management*

1. **Construction Technology.** Engineering Materials : Physical properties of construction materials : Stones, Bricks and Tiles ; Lime, cement and Surkhi Mortars; Lime Concrete and Cement Concrete, Properties of freshly mixed and hardened concrete, Flooring Tiles, use of ferro-cement, fibre-reinforced and polymer concrete, high strength concrete and light weight concrete. Timber : Properties and uses; defects in timber; seasoning and preservation of timber. Plastics, rubber and damp-proofing materials, termite proofing, Materials, for Low cost housing.

Construction. Building components and their functions; Brick masonry : Bonds, jointing. Stone masonry. Design of Brick masonry walls as per I.S. codes, factors of safety, serviceability and strength requirements; plastering, pointing. Types of Floors & Roofs. Ventilators, Repairs in buildings.

Functional planning of buildings : Building orientation, circulation, grouping of areas, privacy concept and design of energy efficient building; provisions of National Building Code. Building estimates and specifications; Cost of works; valuation.

2. **Construction Equipment.** Standard and special types of equipment, Preventive maintenance and repair, factors affecting the selection of equipment, economical life, time and motion study, capital and maintenance cost.

Concreting equipments. Weigh batcher, mixer, vibration, batching plant, Concrete pump.

Earth-work equipment. Power shovel hoe, bulldozer, dumper, trailers, and tractors, rollers, sheep foot roller.

3. **Construction Planning and Management.** Construction activity; schedules, job layout, bar charts, organization of contracting firms, project control and supervision. Cost reduction measures.

Net work analysis. CPM and PERT analysis, Float Times, cashing of activities, contraction of network for cost optimization, up dating, cost analysis and resource allocation.

Elements of Engineering Economics, methods of appraisal, present worth, annual cost, benefit-cost, incremental analysis. Economy of scale and size. Choosing between alternatives including levels of investments, Project profitability.

Part B — *Survey and Transportation Engineering*

Survey. Common methods of distance and angle measurements, plane table survey, levelling traverse survey, triangulation survey, corrections, and adjustments, contouring, topographical map. Surveying instruments for above purposes. Techeometry. Circular and transition curves. Principles of photogrammetry.

Railways. Permanent way, sleepers, rail fastenings, ballast, points and crossings, design of turn outs, stations and yards, turntables, signals, and interlocking, level-crossing. Construction and maintenance of permanent ways : Super elevation, creep of rail, ruling gradient, track resistance, tractive effort, relaying of track.

Highway Engineering. Principles of highway planning, Highway alignments. Geometrical design : Cross section, camber, Super elevation, horizontal and vertical curves. Classification of roads : low cost roads, flexible pavements, rigid pavements. Design of pavements and their construction, evaluation of pavement failure and strengthening.

Drainage of roads : Surface and sub-surface drainage.

Traffic Engineering : Forecasting techniques, origin and destination survey, highway capacity. Channelised and unchannelised intersections, rotary design elements, markings, sign, signals, street lighting; Traffic surveys. Principle of highway financing.

Part C — *Hydrology and Water Resources Engineering*

1. **Hydrology.** Hydrological cycle, precipitation, evaporation, transpiration, depression storage, infiltration, overland flow, hydrograph, flood frequency analysis, flood estimation, flood routing through a reservoir, channel flow routing-Muskingam method.

Ground water flow. Specific yield, storage coefficient, coefficient of permeability, confined and unconfined aquifers, aquifers, aquitards, radial flow into a well under confined and unconfined conditions, tube wells, pumping and recuperation tests, ground water potential.

2. **Water Resources Engineering.** Ground and surface water resource, single and multipurpose projects, storage capacity of reservoirs, reservoir losses, reservoir sedimentation, economics of water resources projects.

3. **Irrigation Engineering.** Water requirements of crops : consumptive use, quality of water for irrigation, duty and delta, irrigation methods and their efficiencies.

Canals : Distribution systems for canal irrigation, canal capacity, canal losses, alignment of main and distributary canals, most efficient section, lined canals, their design, regime theory, critical shear stress, bed load, local and suspended load transport, cost analysis of lined and unlined canals, drainage behind lining.

Water logging : causes and control, drainage system design, salinity.

Canal structures : Design of cross regulators, head regulators, canal falls, aqueducts, metering flumes and canal outlets.

Diversion head work : Principles and design of weirs of permeable and impermeable foundation, Khosla's theory, energy dissipation, stilling basin, sediment excluders.

Storage works : Types of dams, design, principles of rigid gravity and earth dams, stability analysis, foundation treatment, joints and galleries, control of seepage.

Spillways : Spillway types, crest gates, energy dissipation.

River training : Objectives of river training, methods of river training.

Part D — *Environmental Engineering*

1. **Water Supply.** Estimation of surface and subsurface water resources, predicting demand for waters, impurities of water and their significance, physical, chemical and bacteriological analysis, waterborne diseases, standards for potable water.

Intake of water. pumping and gravity schemes. Water treatment : principles of coagulation, flocculation and sedimentation; slow-, rapid-, pressure-, filters; chlorination, softening, removal of taste, odour and salinity.

Water storage and distribution. storage and balancing reservoirs : Types, location and capacity. Distribution system : layout, hydraulics of pipe lines, pipe fittings, valves including check and pressure reducing valves, meters, analysis of distribution systems, leak detection, maintenance of distribution systems, pumping stations and their operations.

2. **Sewerage systems.** Domestic and industrial wastes, storm sewage-separate and combined systems, flow through sewers, design of sewers, sewer appurtenances, manholes, inlets, junctions, siphon. Plumbing in public buildings.

Sewage characterisation. BOD, COD, solids, dissolved oxygen, nitrogen and TOC. Standards of disposal in normal water course and on land.

Sewage treatment. Working principles, units, chambers, sedimentation tanks, trickling filters, oxidation ponds, activated sludge process, septic tank, disposal of sludge, recycling of waste water.

Solid waste. Collection and disposal in rural and urban contexts, management of long-term ill-effects.

3. **Environmental pollution.** Sustainable development. Radioactive wastes and disposal. Environmental impact assessment for thermal power plants, mines, river valley projects. Air pollution, Pollution control acts.

Past Years Questions of Civil Services Competition

The various questions that have been set during the last several years since 1979 in this exam. from the subject of Irrigation and Hydrology, are reproduced here. *You will be glad to notice that many of the numerical questions were directly set from this book.*

Year 1979 (*Civil Services*)

Paper I

Q. 5. *Describe with sketches where necessary how a weir based on Khosla's theory is designed. Discuss the limitations of this theory.*

Q. 8. *(b) A project involves construction of a canal of most economical section with side slopes 2 horizontal to 1 vertical has to discharge 15000 litres per sec with a slope of 10 cm in a kilometre. When unlined C = 45 and when lined C = 60. If the cost of per cub-metre of excavation is one third the cost per sq. metre of lining, which would be cheaper and what would be their dimensions ?*

Year 1980 (*Civil Services*)

Paper I

Q. 5. *Describe briefly various types of spillways and method used for dissipating the energy of the discharge from them.*

Paper II

Q. 11. *Wheat is to be grown in field having a field capacity equal to 27% and the permanent wilting point is 13%. Find the storage capacity in 80 cm depth of the soil, if*

the dry unit weight of the soil is 1.5 gm/cc. If the irrigation water is to be supplied when the average soil moisture falls to 18%, find the water depth required to be supplied to field if the field application efficiency is 80%, what is the amount of water needed at the canal outlet if the water lost in the water course and the field channel is 15% of the outlet discharge ?

[**Note.** Please refer Solved Example 2.14. **This question was set from this book.**]

Q. 12. *An intense rain is assumed to be falling at a uniform rate of 5 cm per hr. for a period of 80 min. on a drainage basin of 550 hectares. The average infiltration capacity during the entire rain period has been worked out to be 1.4 cm per hr. Determine the maximum runoff rate, if the peak percentage based on 10 minutes interval from the distribution graph for this basin is 18%.* (60)

[**Note.** Please refer Solved Example 7.44. **This question was set from this book.**]

Q. 13. *(a) Find the maximum safe width for an elementary profile of a gravity dam of 20 m height. Specific gravity of the dam material is 2.25 and the uplift factor is 0.45.* (20)

(b) Enumerate atleast four types of spillways. Describe any two types in detail illustrated with sketches. (20)

(c) A flood of certain magnitude has a return period of 25 years. What is the probability of exceedence ? What is the probability that this flood may occur in the next 12 years ? (20)

[**Note.** Please refer Solved Example 7.52]

Q. 24. *(a) Discuss the following :*

(i) Wilting point ;

(ii) Evapotranspiration and net Irrigation requirements. (15)

(b) Draw the neat cross-sections of canal in

(i) cutting,

(ii) filing, and

(iii) partial cutting. (15)

(c) Draw a neat L-section of a glacis type fall with baffle wall and platform and clearly label various parts indicating slopes and energy line, etc. (15)

Q. 15. *(a) Briefly discuss the types of rivers.* (15)

(b) Explain with sketches the meandering ratio. (15)

(c) Explain with sketches the theory of river meandering. (15)

(d) Enumerate river training works. Discuss one of these with neat sketches. (15)

Year 1981 (*Civil Services*)

Paper I

Q. 6. Give a brief outline of the procedures for the design of the non-overflow section of a low gravity dam indicating clearly the forces (acting on the dam) being considered for the design. (30)

Paper II

Q. 9. Discuss the following terms bringing out their meaning, significance and write the mathematical equation if any :

(i) Duty of crop

(ii) Delta of a crop

(iii) Optimum utilization of irrigation water

(iv) Consumptive use

(v) Net irrigation requirements ; and

(vi) Field capacity. (60)

Q. 16. A pump is installed on a well to lift water and to irrigate rice crop, sown over a 3 hectares of land. If duty for rice is 864 hectares/cumec on the field and pump efficiency is 48% ; determine the minimum H.P. of the pump if the lowest well water level is 8.0 m below the highest portion of the field. Field channel losses are neglected. (60)

[**Note.** Please refer Solved Example 3.7. **This question was set from this book.**]

Q. 11. Given Q = 50 cumec ; Silt factor = 1.1, side slope = $\frac{1}{2}$: 1, using Laceys theory , determine :

(i) Velocity ;

(ii) area of cross-section ;

(iii) Hydraulic radius ;

(iv) Perimeter, and

(v) Longitudinal slope. (40)

[**Note.** Please refer Solved Example 4.6. **This question was set from this book.**]

(*b*) Discuss regime concept of Lacey. (20)

Q. 12. Results to determine the infiltration capacity equation in the exponential form are tabulated below :

Time in hours	*f in cm/hr*	$f-f_c$	$\log_{10}(f-f_c)$
0	10.4	9.4	0.973 or + 0.973
0.25	5.6	4.6	0.663 or +0.663
0.50	3.2	2.2	0.342 or + 0.342
0.75	2.1	1.1	0.042 or + 0.042
1.00	1.5	0.5	T.699 or (–) 0.301
1.25	1.2	0.2	T.301 or –0.669
1.50	1.1	0.1	T.00 or –1.00
1.75	1.0	0.0,	
2.00	1.0	0.0,	

Plot $\log_{10}(f-f_c)$ against time and determine the infiltration exponential equation. Given that $\log_{10} e = 0.4343$. (60)

[**Note.** Please refer Solved Example 7.13. **This question was set from this book.**]

Year 1982 (*Civil Services*)

Paper I

Q. 6. *The concrete floor of a head regulator is level with the channel bed (except for the short crest hump) and is 13 m long. The floor is provided with cut off walls at its upstream and downstream ends. The depth of upstream cutoff is 1.5 m (below the floor level) and that of the downstream wall is 2.0 m. Using Khosla's theory (see Fig. for definition, sketch and formula), determine the thickness of the floor at its mid-length and also at its junction with the upstream and downstream cutoff walls. The floor thickness may not be less than 30 cm any where. The upstream FSL is 1.5 m above the floor level. If the permissible exit gradient is 0.18, is the floor safe against failure by piping ?*

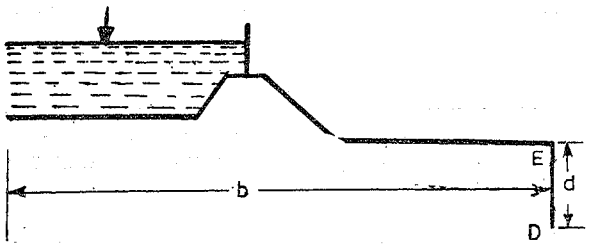


Fig. 31.11. Definition sketch for Khosla's theory for uplift pressure.

$$\phi_E = \frac{1}{\pi}\cos^{-1}\left(\frac{\lambda - 2}{\lambda}\right)$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\alpha = \frac{b}{d}. \qquad (60)$$

[**Note.** Please refer Solved Example 11.4].

Paper II

Q. 9. *Write brief but relevant answers for any three parts :*

(a) Outlining the factors governing runoff, describe their interaction. Also describe briefly the methods for estimating runoff.

(b) Outlining the factors which lead to water-logging and salinity of soils in irrigated areas, discuss the respective remedial measures to reclaim them.

(c) Describe the stages of a river during its course. State the objectives of rive training. Also explain briefly how bunds and spurs function.

(d) Discuss the methods adopted to control seepage through and under-neath earth rock dams. Evaluate their performance.

Q. 10. *(a) Define the duty of a crop. Illustrate the relationship between duty and delta. Also describe how the efficiency of irrigation is determined.*

(b) Describe the procedure for determining the design discharge of an irrigation canal. Discuss why the kor demand of crops should be considered in place of average demand while computing the capacity.

(c) The transplantation of paddy takes 20 days and the total depth of water required by the crop is 25 cm on the field. During the transplantation period, there is about 10 cm of useful rain. Find the duty of irrigation water required for paddy during transplan-

tation period. Also find the duty of water at the head of water course and at the head of the distributary. Assume 25% losses of water in water courses and 15% losses from the distributary head to the water course head.

[**Note.** Please follow Example 3.5. **This question was set from this book by slightly modifying the numerals.**]

Q. 11. *(a) Describe hydrologic cycle and explain its importance.*

(b) Describing the method of determining the yield of a tubewell, discuss the factors contributing to the progressive reduction of its yield.

(c) Explain step by step, the method of construction of unit hydrograph from storm of unit duration stating the basic assumptions.

(d) Discuss general principles governing planned and optimal use of water resources.

Q. 12. *(a) Discuss the various design provisions which are intended to provide additional safety of earth-rock dams against earthquake forces.*

(b) Explain how cracking of embankments leading to piping failure become an important and difficult problem in the present day design of modern earth-rock dams.

(c) Explain how the treatment of foundations including abutments become relevant to the design of hydraulic structures of the most competitive types (namely arch dams and earth-rock dams).

(d) Stating the role of spillways, classify them. Also state the conditions for which each type is appropriate.

Year 1983 (*Civil Services*)

Paper I

Q. 6. (*a*) What is an "economical channel section" ? Work out the conditions for such a channel when the section is trapezoidal having bottom width b, depth d and side slopes l/n.

Part II

Question 9 is compulsory.

Q. 9. Write answers for any three sub-divisions :

(a) Define (i) Delta , (ii) Duty, and (iii) Base period and derive the relationship among them.

Also explain the influence of several factors which affect duty. (20)

(b) Explain the method of estimating maximum flood discharge in a river using unit hydrograph approach. (20)

(c) What is water-logging ? What are the ill-effects of water-logging? How will you reclaim a water-logged area ? (20)

(d) How do you differentiate between a high dam and a low dam ? Explain any two types of spillway in a masonry dam with sketches. (20)

Q. 10. *(a) A reservoir with a live storage capacity of 360 million cubic meters is able to irrigate an ayacut of 40,000 hectares with two fillings each year. If the crop season is 120 days, calculate the duty.* (20)

(b) Explain the isohyetal and the Theiesen's polygon method for finding out the average rainfall of a given basin. (20)

(c) Explain briefly the various factors which affect runoff. Mention the empirical formulae for the estimation of flood flows. (20)

Q. 11. *(a) What do you understand by a regime channel ? Compare the silt theories of Kennedy and Lacey.* (20)

(b) (i) Differentiate between a weir and a barrage. Explain with neat sketches their salient components.

(ii) List the advantages and disadvantages of a Butterss dam. (20)

(c) What are the forces that you will consider while designing a masonry dam in an earthquake region ? How will you account for these forces in the design ? Explain consolidation grouting. (20)

Q. 12. *(a) Discuss the object of the classification of river training work. Draw a practical section of a canal.* (20)

(b) Name the various types of cross-drainage works. Draw neat sketches to show the plan, the cross-section and sectional elevation of these. (20)

(c) What are different ways of aligning a canal ? Where will you use each of them ? Explain with suitable sketches. (20)

Year 1984 (*Civil Services*)

Paper I

Q. 2. *(b) Explain Lacey's regime channel, initial regime and final regime.*

Write the equations as per concept of Lacey for design procedure of a true regime channel carrying a fixed discharge and transporting a given silt grade in an unlimited incoherent alluvium. (20)

Paper II

Q. 9. *Write answers for any three subdivisions :*

(a) What is runoff ? How is it related to the rainfall ? What are the factors that influence the runoff from a catchment ? (20)

(b) Explain how Lacey's theory is an improvement over Kennedy's theory with reference to 'Regime' in channels. What are the steps involved in designing a channel through Lacey's theory ? (20)

(c) How will you reclaim waterlogged land ? Draw neat sketches of (i) carrier drains, and (ii) intercepting drains. (20)

(d) What is Retrogression ? Explain how river weirs are different from river barrages. Explain with neat sketches Syphon well drops and label various parts. (20)

Q. 10. *(a) What are different types of hydroelectric power plant ? Explain the various components of a high head hydroelectric project with a neat layout plan.* (30)

(b) What are the different types of earth dams ? Explain them with neat sketches. (30)

Q. 11. *(a) What are different types of irrigation ? Explain them. Define 'flow duty' and 'quantity duty'.* (15)

(b) Calculate the discharge flowing through a channel of trapezoidal section of bed width 18 m and depth 3 m with $\frac{1}{2}$ to 1 side slopes. The channel has a bed slope of 25 cm for every km and the coefficient of rugosity is 0.03. Given N = 0.9875 and D = 0.8760. (25)

(c) Describe ogee spillway and syphon spillway with neat sketches explaining how

they function. Also explain the general features of energy dissipation devices used in the toe of a spillway. (20)

Q. 12. *(a) List the factors to be considered before selecting a site for constructing a dam. Derive an expression for the elementary profile of a gravity dam in terms of base width and height of the dam.* (30)

[**Note.** Please refer Eqn. 19.23 and Article 19.6.]

(b) Explain the concept of Exit gradient. Also explain the different steps involved in the design of the apron of a weir on permeable foundation using Khosla's theory. (30)

Year 1985 (*Civil Services*)

Paper I

Q. 2. *(a) (i) Explain briefly the method of designing a canal using Lacey theory.*

(ii) Design an irrigation canal in alluvial soil according to Lacey silt theory ; given: Lacey silt factor is 1, canal side slope is 1 horizontal to 2 vertical and full supply discharge is 10 m^3/sec.

[**Note.** Please refer Solved Example 4.9.]

Paper II

Question 9 is compulsory.

Q. 9. *Answer any three sub-divisions :*

(a) Define consumptive use of water by a crop and explain how it is determined under natural conditions. (20)

(b) Describe how a unit hydrograph is prepared from an isolated storm. List and evaluate the basic assumptions. (20)

(c) Explaining waterlogging and salinity of soils in irrigated areas, discuss the causative factors and reclamation process. (20)

(d) Explain why earth-rock dams are increasingly adopted in preference to other types of dams. (20)

Describe the nature of foundation problems in hydraulic structures and their treatment. Illustrate the answer citing case histories. (20)

Q. 10. *(a) The figures below represents streamflow in cumecs from a 1000 acre drainage area. Construct a unit hydrograph for this basin.*

Time (hrs)	2	4	6	8	10	*N noon*	2	4	6
Flow (cumecs)	5.1	6.5	7.4	10.2	8.8	7.4	5.9	5.4	4.8

[**Note.** Please refer Solved Example 7.39]. (25)

(b) Describing the isohyetal method for finding average rainfall over a drainage basin, bring out its merits relative to station average method. (15)

(c) Discuss the factors that affect the runoff from a catchment. (10)

(d) Describe the design consideration of under-sluices provided in diversion head works. (10)

Q. 11. *(a) Construct a mass diagram for the stream having the following monthly discharges during the 2 year period of record. What is the reservoir size required to provide a constant downstream flow of 3 cumecs?*

Month	Monthly discharge ha.m	
	In 1st year	In 2nd year
January	57.4	10.2
February	65.5	30.8
March	28.6	43.1
April	32.8	53.1
May	36.9	38.9
June	24.6	28.9
July	10.2	16.4
August	2.1	12.3
September	2.1	12.3
October	2.1	4.1
November	4.1	18.2
December	8.2	2.1

(30)

(b) Which slope (s) (upstream or downstream or both) of a zoned earth dam is (are) analysed for the three independent conditions ? Name the three conditions and state with reason the normal safety factor recommended for each condition. (9)

(c) State the design criteria for filter material. (6)

(d) Classify spillways stating the conditions favourable for each type. Evaluate the merits and demerits of syphon spillway. (15)

Q. 12. *The command area of a canal is planted with ragi. The base period for the crop is 120 days and delta is 50 cm. If 33% of water is lost in conveyance, compute the area which can be irrigated per 100 Ips let into the canal from the reservoir. If the actual amount of evapotranspiration of the crop is 35 cm. compute the overall irrigation efficiency.* (20)

(b) Describe how the longitudinal section of an irrigation canal is fixed. (8)

(c) Discuss Lacey's theory of flow in alluvial canals. (10)

(d) Describe how the yield of a well is determined by a recuperation test. (10)

(e) Describe the method of analysis of gravity dam and demonstrate how the stability is enhanced by foundation treatment (grout curtain and drainage wells.) (12)

Year 1986 (*Civil Services*)

Paper-I

Q. 5. *(a) What is a canal outlet ? Explain the requirements to be satisfied by a canal outlet. Distinguish between a hyper-proportional outlet and a sub-proportional outlet.* (30)

(b) Water flows over a concrete spillway at the rate of 300 cumecs. After flowing over the spillway it passes over a level concrete apron (n = 0.015). The velocity of water at the toe of the spillway is 15 m/sec. and the width of the apron is 50 m. The tail water depth is 3 m. Calculate the length of the apron to contain the jump. Also calculate the energy dissipated. (30)

Q. 6. *(a) Explain the effects of construction of a weir on the regime of a river. Explain with neat sketches the working of a silt excluder.* (30)

(b) Write an explanatory note on filter criteria for earthen dams. Explain the method of checking the stability of earth dam foundation against shear failure. (30)

Paper II

Q. 9. *Attempt any three sub-divisions :*

(a) Derive Seddon's law using both Chezy's equation and Manning's equation for the passage celerity of a flood wave in a wide rectangular open channel. (20)

(b) (i) What is meant by the basic profile of a low gravity dam ? (5)

(ii) Develop the basic profile of a low concrete gravity dam where the specific gravity of the concrete of the dam is 2.5, and uplift is credited with an intensity factor of 0.5 and an area factor also of 0.5. Allowable vertical compressive stress (in the presence of shear stresses — i.e. the vertical stress is not the principal stress necessarily) is 8.5 kg/cm^3. (15)

(c) Bring out features of the hydrological cycle indicating the interrelationships also. All major stages or phases should be clearly emphasized including the anomalies to be cautioned against. (20)

(d) What are the major components of a canal irrigation system ? What are the functions that each performs ? What are the main appurtenances in, across, or along each component ? (20)

Q. 10. *(a) (i) Mention any three objectives for which river training works are constructed. What are the different types of works or constructions adopted in this connection ?* (12)

(ii) Draw a typical plan view of a Bell's guide bund showing the flow direction, the angles and the nominal lengths also; and name the several parts and locations. (8)

(b) (i) Mention six different types of spillways. Also mention four different types of gates used on spillways. (5)

(ii) What are the several features that affect the hydraulic design of an ogee spillways ? (15)

(c) Design a channel by Lacey's theory for 40 cumec capacity. The side slopes may be assumed to be 1 : 1. The average size of the bed material may be taken as 0.8 mm. (20)

Q. 11. *(a)(i) A canal takes off from a reservoir to irrigate the areas as shown in the table. 40% of the water required for irrigation is assumed to be available directly from precipitation. Channel conveyance losses are 15%. Reservoir losses are 10%. What would be the capacity of the reservoir needed ? (The reservoir is to be filled only once a year).* (12)

Crop	*Base period, days*	*Duty of the field, ha/cumec*	*Area under crop, ha*
Wheat	*120*	*1800*	*500*
Sugarcane	*320*	*800*	*600*
Rice	*120*	*900*	*300*
Cotton	*200*	*1400*	*1200*
Bajra	*100*	*1200*	*500*

[Ans. 2890 ham]

(ii) What are the advantages of the sprinkler system of irrigation ? (8)

(b) Derive the Dupuit's equation for the centre-to-centre spacing of a circular tile drains battery below ground for alleviating water-logging. You can take L as the spacing between the drains forming the drainage system, a as the height of the pipe system above the impervious layer, b as the maximum height of the watertable above the impervious layer and q as the rate of discharge per unit area of land surface. (20)

(c) An unregulated stream provides the following volumes through each successive 4-day period over a 40-day duration at a possible reservoir site. What would be the reservoir capacity needed to ensure maintaining the average flow for these 40 days if

Day	0	4	8	12	16	20	24	28	32	36	40
Runoff volume, $m^3 \times 10^6$	0	9.6	5.4	2.3	3.5	2.3	2.2	1.4	6.4	12.4	10.9

the reservoir is full to start with? What is this average flow ? What would be the approximate quantity of water wasted in spillage in this case ? (20)

Q. 12. *(a) Discuss the several relative conditions that can occur below spillways by the comparative locations of the tail water rating curve and the jump height curve. What are the methods adopted for efficient energy dissipation in each case ? By what mechanism is energy dissipated in each method ?* (30)

(b) (i) The Muskingum method by McCarthy assumes the reach storage in a stream to be given by $S = K[xI + (I - x)O]$, where K is the storage constant. Also, the basic routing equation written for discrete time is,

$$\frac{I_1 + I_2}{2} t - \frac{O_1 + O_2}{2} t = S_2 - S_1$$

Derive from these the Muskingum equation and incidentally determine the coefficients therein. What is the sum of the coefficients ? (10)

(ii) The storage in a stream reach has been studied ; and x and K have been identified as 0.28 and 1.6 days. If the inflow hydrograph in the stream reach, as the flood starts coming in and passes, is given by the following table, compute the outflow hydrograph. (Plotting is not needed) :

Hours	0	6	12	18	24	30
I m^3/sec	35	55	92	130	160	140

(10)

[**Note.** Please refer Solved Example 13.3 in "Hydrology and Water Resources Engineering".]

Year 1987 (*Civil Services*)
Paper I

Q. 5. For the following data, make necessary hydraulic calculations for a vertical drop weir using the Bligh's theory :

Maximum flood discharge	= *2800 cumecs*
HFL before construction	= *285.0 m*
Minimum water level	= *278.0 m*
Downstream bed level	= *278.0 m*
F.S.L. of canal	= *284.0 m*
Allowable afflux	= *1 m*
Coefficient of creap	= *12*
Permissible exit gradient	= *1/6*

(60)

Q. 6. *(a) Explain the different types of spillways used in dam construction. State briefly the design principles of an ogee spillway in sequence.* (30)

(b) Design an irrigation canal for a discharge of 15 cumec, N = 0.225, m = 1 and $\frac{B}{D} = 5.7$. *Use the Kennedy theory.* (30)

Paper II

Q. 9. *Attempt any three subdivisions :*

(a) Derive the basic probability of an N-year event defined in the conventional way ; indicate the definition also. Accordingly, derive the probability of a flood with a return period of 20 years, occurring in a particular 3-year period. (20)

(b) (i) What are the factors (list at least six) that affect the pattern of sediment deposition in a reservoir. (10)

(ii) A proposed reservoir has a capacity of 50 ha-m. The catchment area is 125 km^2 and the annual streamflow averages 12 cm of run-off. If the annual sediment production is 0.03 ha-m/km^2, what is the probable life of the reservoir before its capacity is reduced by 10% of its initial capacity by sedimentation ? The relationship between trap efficiency, n (%) and capacity-flow ratio, C/I , is as under :

C/I :	0.01	0.02	0.04	0.06	0.08	0.1	0.2	0.3	0.5	0.7
η (%) :	43	60	74	80	84	87	93	95	96	97

[**Note.** Please refer Solved Example 18.16]. (20)

(c) What are the several natural and man-made causes of water-logging ? What are the ill-effects due to water-logging? What are the several measures practicable for minimising water-logging effects ? (20)

(d) What are the purposes for which foundation treatment for hydraulic structures is undertaken ? What are the several features that are treated in this manner ? Write briefly on each. (20)

Q. 10. *(a) Classify the common methods of irrigation and indicate the suitability and advantages of each main method besides indicating how each method is practised.* (30)

(b) How are river training works classified ? What is the propose achieved in each of these classes of training works and how is it achieved? (15)

(c) Define the following : (i) Alkali soil, (ii) Alkaline soil, (iii) Leaching, Sodium absorption ratio. (15)

Q. 11. *(a) A multi-purpose reservoir for providing irrigation water, flood control and low-flow augmentation for water quality control is under consideration. The following curves have been generated by proper analysis : (i) Reservoir cost curve, (ii) Reservoir yield curve, (iii) Annual irrigation demand, (iv) Annual flood control benefits, (v) Water quality benefits from low-flow augmentation and (vi) Water requirement for low augmentation. Draw the general shape of each of these curves indicating the entities represented on the coordinate axes.* (25)

(b) A project has been assessed for different levels of investment and is to be evaluated for a 10 year life at 10% return on its capital. The consecutive levels of

investment and the corresponding net annual returns are assessed to be as under (all in lakhs of rupees) :

Investment :	*500*	*600*	*700*	*800*	*900*	*1000*
Annual returns :	*100*	*110*	*122*	*132*	*141*	*149*

Considering the appropriate aspects of evaluation, determine how large an investment should be made. SFF for 10 years at 10% = 0.06275. (20)

(c) Calculate the maximum discharge required at the outlet of a distributary having the following cropped area and duties of water : (15)

Crop	*Area (in ha)*	*Crop Season*	*Duty (in '000 ha/cumec)*
Sugarcane	*200*	*Throughout*	*0.730*
Other perennials	*1000*	*Throughout*	*1.100*
Hot weather fodder	*3000*	*Hot weather*	*2.000*
Rice	*1500*	*Kharif*	*0.775*
Other Kharif	*1500*	*Kharif*	*1.500*
Rabi	*5000*	*Rabi*	*1.800*

Q. 12. *(a) A concrete gravity dam has the following design features :*

Top of dam	*= 200 m RL*
Width at top	*= 7 m*
Bottom of top rectangular position	*= 190 m RL*
Downstream battery	*= 0.7 h on 1 v*
Upstream battery is below 150 m RL and has a value	*= 1 h on 10 v*
Drainage gallery at 10 m from u/s vertical face ; Uplift reduction at gallery	*= 0.6*
Tail water depth	*= 5 m*
Wt. of concrete	*= 2.4 T/m^3*
Water stands upto	*= 195 m RL*
Base of dam	*= 100 m RL*

Determine the vertical normal stresses at each end of the base when the reservoir is full as indicated. (30)

(b) What are the criteria for safe design of earth dams ? (15)

(c) A storm with a 15.0 cm precipitation produced a direct runoff of 8.7 cm. The time distribution of the storm is as follows :

Time from start hr.	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*
Incremental rainfall in each hour, cm	*0.6*	*1.35*	*2.25*	*3.45*	*2.7*	*2.4*	*1.5*	*0.75*

Estimate the ϕ_{index} of the storm. (15)

[**Note.** Please refer Solved Example 7.26.]

Year 1988 (*Civil Service*)

Paper I

Q. 5. *(a) A concrete dam 30m high retains water to a maximum height of 25 m on upstream side. The upstream face is vertical for the upper 10 m of the dam and then sloping towards upstream at 1 horizontal to 10 vertical up to the base. The top width*

of the dam is 4 m for a height of 5 m and then splays in the downstream giving a total base width of 20 m. There is tail water for a depth of 5 m and the axis of drainage gallery is at 4 m from the heel. Assuming the specific gravity of concrete as 2.4, calculate the factor of safety against overturning and the stress at the toe and the heel under reservoir full condition. (30)

(b) Describe briefly the function and design of a silt excluder provided in a diversion head work. Supplement your answer with neat sketches. (30)

Q. 6. *(a) Design the section of an unlined canal in a loamy soil to carry a discharge of 50 cum/sec with a permissible velocity of 1 m/sec. Assume the side slopes as 2 : 1 and B/D ratio as 6.0. Using Manning formula, calculate the bed slope of the canal, N = 0.0225.* (15)

(b) Discuss briefly the different types of failure of earth dams and measures to prevent them. (30)

(c) Explain the technical and financial justification for lining of existing canals. (15)

Paper II

(Part C — *Water Resources Engineering*)

Q. 9. *(Compulsory) Answer any three subdivisions :*

(a) What is runoff ? Discuss the factors influencing the runoff of a catchment area. (20)

(b) What is duty of water ? Explain the influence of several factors affecting duty. (20)

(c) Enumerate the different methods of river training. Describe the method of marginal embankments. (20)

(d) Discuss the factors influencing the selection of a site for a proposed dam. (20)

Q. 10. *(a) What is unit hydrograph ? How is it obtained ? Explain its use in the construction of flood hydrograph resulting from rainfalls.* (20)

(b) Explain flood routing through reservoirs. Describe stepwise the procedure adopted for flood routing computations by the trial and error method. (20)

(c) Write explanatory notes on the different types of irrigation. (20)

Q. 11. *(a) Explain the governing considerations for fixing up the alignment, the full supply level, and the full supply discharge of an irrigation canal.* (20)

(b) The gross command area for a distributary is 10,000 hectares, 75% of which can be irrigated. The intensity of irrigation for Rabi season is 60% and that for Kharif season is 30%. If the average duty at the head of the distributary is 2500 hectares per cumec for Rabi season, and 1000 hectares per cumec for Kharif season, determine the discharge required at the head of the distributary from average demand consideration. (20)

[Note. Please refer and follow Example 3.1 and determine the requisite discharge as 2.25 cumecs. **Ans.]**

(c) Explain the principal causes and the effects of water-logging in a canal irrigated farm. State the precautions and measures adopted to prevent water-logging of irrigated lands. (20)

Q. 12. *(a) State the general conditions of stability of gravity dams. Explain stepwise the procedure of analysing high gravity dams.* (20)

(b) Enumerate the different types of earth dams indicating the situation where each type is more suitable. Explain in sequence the steps involved in determining the stability of earth dams by the Swedish slip circle method. (20)

(c) State the governing considerations for the selection of different types of spillways. Explain the design principles of ogee spillway. (20)

Year 1989 (*Civil Services*)

Paper I

[**Note.** ***Due to revision in syllabus w.e.f. 1989, Paper I did not contain any question on Irrigation, as it contained questions on Structural Engg., Fluid Mechanics, and Soil Mechanics only, in 3 sections.***]

Paper II

(**Part C** — *Water Resources and Irrigation Engineering*)

Q. 9. *(Compulsory). Attempt any three sub-divisions :*

(a) (i) Explain briefly what you understand by the terms :

1. Design flood

2. Standard project flood

3. Probable maximum flood

4. Design storm. (10)

(ii) What are the recommended design floods for :

1. Spillways of major projects

2. Permanent barrages

3. Waterways of aqueducts

4. Pick-up weirs. (10)

(b) (i) Describe briefly the factors affecting the evaporation from water bodies. Describe a commonly used evaporimeter. (10)

(ii) An isolated 3 hour storm occurred over an area of 120 ha as below:

Partial area of catchment (ha)	*ϕ index (cm/h)*	*Rainfall (cm)*		
		1st hour	*2nd hour*	*3rd hour*
36	*0.90*	*0.6*	*2.4*	*1.3*
18	*1.10*	*0.9*	*2.1*	*1.5*
66	*0.50*	*1.0*	*2.0*	*0.9*

What is the total rainfall on the catchment in this storm ? Estimate the runoff from the catchment. If the ϕ index were to be remain at the same value, what runoff would be produced by a uniform rainfall of 3.3 cm in 3 hours uniformly spread all over the catchment ? (10)

[**Note.** Please refer Solved Example 7.28].

(c) Describe briefly the forces considered in the design of gravity dams: what combination of these forces is critical in the design of non-overflow section of the dam under reservoir empty condition ? (20)

(d) (i) Explain very briefly the following terms used in groundwater flow :

1. Specific yield

2. Storage coefficient

3. Barometric efficiency

4. Specific capacity. (10)

(ii) Write short notes on :

1. Irrigation water quality

2. Leaching. (10)

Q. 10. *(a) What are the advantages of lining a canal ? Explain how the benefit cost ratio for lining an existing canal can be estimated. Indicate clearly the assumptions, if any, involved.* (15)

(b) Write brief notes on :

(i) Bed load

(ii) Suspended load

(iii) Critical shear stress in sediment transport. (15)

(c) The following table gives the monthly inflow into and contemplated demand from a proposed reservoir :

Month	*Jan.*	*Feb.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sept.*	*Oct.*	*Nov.*	*Dec.*
Monthly inflow $M.m^3$	50	40	30	25	20	30	200	225	150	90	70	60
Monthly demand $M.m^3$	70	75	80	85	130	120	25	25	40	45	50	60
Evapo-ration, cm	6	8	13	17	22	22	14	11	13	12	7	5
Rainfall cm	1	0	0	0	0	19	43	39	22	6	2	1

Assume the average reservoir area as 30 km^2. The runoff coefficient of the area flooded by the reservoir is 0.4. Estimate the minimum storage that is needed to meet the demand. (30)

[**Hint.** Follow Example 8.2 and compute the reqd. storage capacity = 133.08 ha. m, as per computations carried out in table given on next page. **Ans.**]

Q. 11. *(a) (i) Distinguish between :*

1. Hydrologic and hydraulic methods of flood routing.

2. Hydrologic storage routing and hydrologic channel routing. (10)

[**Hint.** Flow routing is a procedure by which we can estimate/trace the flow through a *hydrologic system* (like a river or a reservoir, or a canal), downstream of a given input point. This can be done either by adopting *hydrologic routing techniques* or by *hydraulic routing techniques.*

In **hydrological routing,** the flow is calculated as a *function of time alone at a particular location* ; whereas in a **hydraulic routing,** *the flow is calculated as a function of space and time both,* throughout the system.

Hydrologic storage routing has been adopted by us in Chapter 18, to compute the levels likely to be achieved in a reservoir due to entry of a flood hydrograph, and to work out the outflow hydrograph, spilling over the spillway. *Hydrologic channel routing* has similarly been used in Muskingum's method for determining the shape of flood hydrograph at a particular known point along a river channel downstream of a given input point upstream, as explained in Chapter 13 of "Hydrology and Water Resources Engineering".

Solution Table for Q. 10 (*c*) of 1989 (Civil Services)

Month	*Inflow in Mm^3*	*Evaporation cm*	*Rainfall in cm*	*Demand Mm^3*	*Evaporation loss* $= \frac{30\ km^2 \times col.\ (3)}{100}\ m$ $= 0.3 \times col.\ (3)$ *in $M.m^3$*	*Rainfall in Mm^3; only 60% rainfall is now counted, because 40% was already counted as runoff in the observed inflows before construction of dam* $= \frac{0.6 \times col.\ (4)}{100}\ m \times 30\ km^2$ $= 0.18 \times col.\ (4)$	*Adjusted inflow in $M.m^3$ col. (2) – col. (6) + col. (7)*	*Water reqd. from storage in $M.m^3$ col. (5) – col. (8) (only +ve values)*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*
Jan.	50	6	1	70	1.8	0.18	48.38	21.62
Feb.	40	8	0	75	2.4	0.00	37.60	37.4
March	30	13	0	80	3.9	0.00	26.10	53.9
April	25	17	0	85	5.1	0.00	19.90	5.10
May	20	22	0	130	6.6	0.00	13.70	6.30
June	30	22	19	120	6.6	3.42	26.82	3.18
July	200	14	43	25	4.2	7.74	203.54	Nil
August	225	11	39	25	3.3	7.02	228.72	Nil
Sept.	150	13	22	40	3.9	3.96	150.06	Nil
Oct.	90	12	6	45	3.6	1.08	87.48	2.52
Nov.	70	7	2	50	2.1	0.36	68.26	1.74
Dec.	60	5	1	60	1.5	0.18	58.68	1.32

Reqd. storage from reservoir = Σ = 133.08 Mm^3. **Ans.**

The **hydraulic flood routing techniques,** such as those by the use of *Saint Venant Equations,* and *Dynamic wave routing,* involve complicated mathematics, and is generally used only when flows are to be analysed on a computer model. Such hydraulic routing models prove better than the hydrologic models in representing the actual unsteady non-uniform nature of flow in the channel, and also give reliable results even for routing *smaller flows,* such as for routing water in irrigation canals.]

Q. 11. *(a) (ii) Route the following flood through a river reach for which the Muskingum coefficients K and X are 22 h and 0.25 respectively. At time t = 0, the outflow discharge is 40 m^3/s :*

Time (h)	*0*	*12*	*24*	*36*	*48*	*60*	*72*
Inflow (m^3/s)	*40*	*65*	*165*	*250*	*240*	*205*	*170*

(10)

[**Note.** Please refer Solved Example 13.4 in "Hydrology and Water Resources Engineering".]

Q. 11. *(b) Describe the salient features and design considerations of the hydraulic jump type energy dissipators used below overflow spillways.* (20)

Q. 11. *(c) Describe the functions of 'Distributary Head Regulators' and 'Cross Regulators'. Sketch a typical section of a Distributary Head Regulator and indicate the salient parts. List the design principles for a Distributary Head Regulator.* (20)

Q. 12. *(a) (i) A weir across an alluvial river has a horizontal floor of length 60 m and retains 6 m of water under full pond condition. If the downstream sheet pile is driven to a depth of 6 m below the average bed level, calculate the exit gradient. Further, assuming a porosity of 30% and the relative density of soil particles as 2.70, estimate the critical exit gradient and the factor of safety of the system with respect to the exit gradient.* (10)

(ii) Write explanatory notes on the following terms connected with a barrage :

(1) Scouring sluices

(2) Divide wall

(3) Downstream loose protection. (15)

Q. 12. *(b) List the various objectives of river training and indicate against each objective the methods of training that can be adopted to achieve the same.* (20)

Q. 12. *(c) List the essential requirements of an ideal irrigation outlet. Describe with the aid of sketches, the open flume semi-modular outlet.* (15)

Year 1990 (*Civil Services*)

Q. 9. *(Compulsory) Attempt any three sub-divisions :*

(a) Describe the salient characteristics of precipitation in India. (20)

(b) A certain crop is grown in an area of 3000 hectares, which is fed by a canal system. The data pertaining to irrigation are as follows :

Field capacity of soil	*= 26%*
Optimum moisture	*= 12%*
Permanent wilting point	*= 10%*
Effective depth of root zone	*= 80 cm*
Apparent relative density of soil	*= 1.4*

If the frequency of irrigation is 10 days and the overall irrigation efficiency is 22%, find :

(i) the daily consumptive use ; and

(ii) the water discharge in m^3/s required in the canal feeding the area. (30)

[Hint. Follow example 2.13 and 2.14, and determine :

Depth of water used by plants for growth, which is supplemented by irrigation after every 10 days

$$= \frac{\gamma d}{w} [\text{Field capacity m.c.} - \text{Optimum m.c.}]$$

$$= \frac{1.4 \times 0.8 \text{ m}}{1} \left[0.26 - 0.12\right] = 0.1568 \text{ m} = 15.68 \text{ cm.}$$

(*i*) Daily water consumed by plants ; *i.e.*, daily consumptive use

$$= \frac{15.68}{10} \text{ cm} = 1.568 \text{ cm.} \quad \textbf{Ans.}$$

(*ii*) Total irrigation water required *i/c* losses in field and conveyance

$$= \frac{\text{N.I.R.}}{\eta_{irrigation}} = \frac{1.568}{0.22} \text{ cm/day} = 7.127 \text{ cm/day}$$

Field area $= 3000 \text{ ha} = 3000 \times 10^4 \text{ m}^2$

$\therefore$ Discharge required in irrigation canal

$$= \frac{7.127}{100} \times \frac{3000 \times 10^4}{24 \times 60 \times 60} \text{ m}^3/\text{s} = \mathbf{24.75 \text{ m}^3/\text{s}} \quad \textbf{Ans.]}$$

(c) Describe the causes and effects of water-logging in canal irrigated areas. Indicate the measures to be adopted to prevent water-logging. (20)

(d) Write short notes on :

(i) Reservoir sedimentation.

(ii) Silt ejectors.

(iii) Guide bunds.

(iv) Well-loss. (20)

Q. 10. *(a) For a river, the estimated flood peaks for two return periods by use of Gumbel's method are as follows :*

Return period (years)	*Peak flood (m^3/s)*
100	*485*
50	*445*

What flood discharge in this river will have a return period of 1000 years. (30)

[Note. Please refer Solved Example 7.57.]

(b) Write short notes on :

(i) Launching apron *(ii) Groynes*

(iii) Silt excluder *(iv) Recuperence test for a well*

(v) Piping failure of a weir floor *(vi) Tainter gate.* (30)

Q. 11. *(a) An impervious floor of a weir on permeable soil is 16 m long and has sheet piles at both the ends. The upstream pile is 4 m deep and the downstream pile is 5 m deep. The weir creates a net head of 2.5 m. Neglecting the thickness of the weir floor, calculate the uplift pressures at the junction of the inner faces of the pile with the weir floor, by using Khosla's theory.* (30)

[**Note.** Please refer Solved Example 11.3].

(b) Describe briefly the energy budget method of estimating evaporation from a lake. (20)

(c) Describe with the help of a neat sketch the salient features of a Sarda type canal fall. (10)

Q. 12. *(a) Write short notes on :*

(i) Maximum-Depth-Area-Duration Curves.

(ii) Probable Maximum Precipitation

(iii) Potential Evapotranspiration.

(iv) Infiltration process. (20)

(b) A commonly used shape for concrete lined canals, called the standard lined trapezoidal canal section, consists of a trapezoidal section with each of its two corners being rounded off by a circular arc of a radius equal to the full supply depth. Determine the dimensions of such a standard lined trapezoidal canal to carry a discharge of 200 m^3/s at a bed slope of 1 in 4000. The side slopes are 2 horizontal : 1 vertical, and Manning's n = 0.014. The limiting velocity in the canal is 2.0 m/s. (20)

[**Hint.** Follow exactly similar solved example 5.4 and determine

$$R = 2.66 \text{ m}; \quad \cot\theta = 2; \quad \theta = 0.46 \text{ radians.}$$

$$A = 100 = y\,[B + 0.46\,y + 2y]$$

or $$B = \left[\frac{100}{y} - 2.46\,y\right] \quad \ldots(i)$$

Also $$R = 2.66 = \frac{100}{P}, \text{ where}$$

$$P = B + 0.92y + 2y \times 2 = \frac{100}{2.66}$$

or $$B = 37.59 - 4.92y \quad \ldots(ii)$$

Solving (*i*) and (*ii*)

$$\left.\begin{array}{l} y = 3.44 \text{ m} \\ B = 20.7 \text{ m.} \end{array}\right] \quad \textbf{Ans.}$$

(c) Explain briefly a procedure for calculating the stability of slopes of an earth dam. What are the critical loading conditions at which the stability of an earthen dam is tested during design ? (20)

Year 1991 (*Civil Services*)

Q. 9. *Attempt any three sub-divisions.*

(a) Describe briefly the ground water resources of India and its utilization. (20)

(b) Explain how seismic forces are taken into account in the design of solid gravity dams. (20)

(c) (i) Describe briefly the dilution technique of stream flow measurement. (10)

(ii) A trapezoidal channel with side slopes 1.5 horizontal : 1 vertical is required to carry 15 m^3/s of flow with a bed slope of 1 in 4000. If the channel is lined, the Manning's coefficient n will be 0.014, and it will be 0.028 if the channel is unlined. Calculate the average boundary shear stress if hydraulically efficient lined channel is adopted. What percentage of earthwork is saved in a lined section-relative to an unlined section, when hydraulically efficient section is used in both the cases ? The freeboard can be assumed to be 0.75 m in both the cases and the lining can be assumed to be up to the top of the section. (10)

[**Note.** Please refer Solved Example 4.4]

(d) Write short notes on :

(i) Drip irrigation

(ii) Bed load

(iii) Meanders (20)

Q. 10. *(a) List the different types of spillway crest gates in common use. Describe with neat sketches the salient features of radial gates and vertical lift gates.* (30)

(b) (i) What are the assumptions in the unit hydrograph theory

(ii) The ordinates of 4-hour hydrograph of a catchment are given below :

Time (h)	0	4	8	12	16	20	24	28	32	36	40	44
Ordinate of 4-h U.H. (m^3/s)	0	20	60	150	120	90	70	50	30	20	10	0

Derive the flood hydrograph in the catchment due to the storm given below :

Time from start of storm (h)	0	3	8	12
Accumulated rainfall (cm)	0	5.0	5.8	8.8

The ϕ-index for the catchment can be assumed to be 0.25 cm/h and a constant base flow of 20 m^3/s be appropriate. (25)

[**Note.** Please refer Solved Example 7.37].

Q. 11. *(a) Explain briefly the concept of critical shear stress as used in sediment transport studies. Sketch and label the salient features of Shields curve relating to the critical shear stress. Show that in an unlined canal of hydraulic radius R and bed slope, S_0 constructed in a non-cohesive material, the minimum size d of the sand grain that will not be moved by the flow could be expressed as $d = 11\ R.S_0$.* (20)

(b) With reference to a water resources development project, explain briefly :

(i) Engineering economy study

(ii) Project formulation. (30)

[**Note.** Please refer chapter 20 in "Hydrology and Water Resources Engineering"].

(c) Explain briefly the guidelines currently used in India for selection of design floods for dams and barrages. (10)

Q. 12. *(a) A well is located in 30 m thick confined aquifer of permeability 35 m/day and storage coefficient of 0.004. If the well is pumped at the rate of 1500 litres per minute, calculate the drawdown at a distance of 40 m from the well after 20 hours of pumping.* (20)

[**Note.** Use Jacob's formula and compute drawdown as 0.94 m. **Ans.**]

(b) Describe the use of hydraulic jump as an energy dissipator on the downstream side of overflow spillways. (25)

(c) In a wide stream, a suspended load sample taken at a height of 0.30 m from the bed indicated a concentration of 1000 ppm of sediment by weight. The stream is 5.0 m deep and has a bed slope of 1/4000. The bed material can be assumed to be of uniform size with a fall velocity of 2.0 cm/s. Estimate the concentration of the suspended load at mid-depth. (15)

[**Note.** Please refer Solved Example 4.10].

Year 1992 (*Civil Services*)

Q. 9. *(Compulsory) Answer any three of the following :*

(a) Discuss the considerations to be made for selection of site of rain gauge stations.

How would you estimate the missing rainfall data at a rain gauge station for a particular rain storm by Normal Ratio method ? Under what conditions is it adopted ? (20)

(b) Explain the terms duty, delta, and base period in connection with irrigation. Establish a relation between Duty of water, Delta and Base period.

Discuss the factors on which duty of water depends. (20)

(c) Discuss the relative advantages and disadvantages of a Barrage over a Weir. Explain the functions of Undersluices in a barrage. (20)

(d) Draw a typical cross-section of a masonry dam. Briefly explain the principle of design of this cross-section. Explain the functions of drainage gallery. (20)

Q. 10. *(a) What is unit hydrograph ? Discuss the assumptions involved in Unit Hydrograph Concept.* (20)

(b) Discuss in details the general procedure of deriving a unit hydrograph. Explain your answer with a neat diagram. (20)

(c) Name some statistical methods of forecasting peak flood discharge. Describe any one of these methods : (20)

Q. 11. *(a) Explain the terms :*

(i) Specific yield (ii) Coefficient of transmissibility, (iii) Storage coefficient, (iv) Infiltration gallery. (20)

(b) A well penetrates into an unconfined aquifer having a saturated depth of 15 m.

The discharge is 8000 litres/hour at 6 m drawdown. Assuming equilibrium flow conditions and a homogeneous aquifer, compute the discharge at 9 m drawdown. The radius of influence may be taken as equal in both cases. Deduce the formula that you would use. (20)

Solution. Using equation 16.17, we have

$$Q = \frac{\pi K (d^2 - h_w^2)}{2.3 \log_{10} \frac{R}{r_w}}$$

where $h_w = d - s$

where d = saturated depth of stratum

s = drawdown

h_w = water head in well.

$$\therefore \quad Q_1 = \frac{\pi K (d^2 - h_{w1}^2)}{2.3 \log_{10} \frac{R}{r_w}} \quad \text{and} \quad Q_2 = \frac{\pi K (d^2 - h_{w2}^2)}{2.3 \log_{10} \frac{R}{r_w}}$$

$$\therefore \quad \frac{Q_2}{Q_1} = \frac{(d^2 - h_{w2}^2)}{(d^2 - h_{w1}^2)} = \frac{d^2 - (d - s_2)^2}{d^2 - (d - s_1)^2}$$

Using given values of $Q_1 = 8000\ L/h$, $s_1 = 6$ m, $d = 15$ m and $s_2 = 9$ m, we get

$$Q_2 = \frac{15^2 - (15 - 9)^2}{15^2 - (15 - 6)^2} \times Q_1 = \frac{15^2 - 6^2}{15^2 - 9^2} \times 8000\ L/h$$

$$= \frac{21 \times 9}{24 \times 6} \times 8000\ L/h = 10{,}500\ L/h. \quad \textbf{Ans.}$$

(c) Describe briefly the ground water resources of India and its utilization. (20)

Q. 12. *(a) What are the different types of alignment of irrigation canals? Mention the situation where each type is suitable.* (15)

(b) Explain the phenomenon of hydraulic jump. Discuss its use in the design of hydraulic structures.

(c) Write short notes on :

(i) Bed load and suspended load.

(ii) Critical shear stress in sediment transport.

(iii) Guide-bunds.

(iv) Launching apron.

(v) Piping failure of a weir floor. (5 × 6)

Year 1993 (*Civil Services*)

Q. 9. *(Compulsory) Answer any three of the following :*

(a) What are the objectives of command area development ? Explain how these may be achieved. (20)

(b) (i) What is meant by consumptive use of water for a crop ? On what factors does this depend ? (10)

(ii) During a particular stage of the growth of a crop, consumptive use of water is 2.5 mm/day. Determine the interval in days between irrigations, and the depth of water to be applied when the amount of water available in the soil is 50% of the maximum depth of available water in the root zone, which is 80 mm. Assume irrigation efficiency to be 60%. (10)

[Solution. *Study Solved Example 2.13, and compute :*

Readily available moisture in root zone

$$= 50\% \times 80 \text{ mm} = 4 \text{ cm}.$$

Consumptive use = 2.5 mm/day = 0.25 cm/day

$\therefore$ 0.25 cm of water is consumed by crop in = 1 day

4 cm of water will be consumed by crop in

$$= \frac{1}{0.25} \times 4 = \mathbf{16 \text{ days}}.$$

Also, the depth of water to be recouped through irrigation = 4 cm.

Actual water required to be applied to the field

$$= \frac{\text{Water required to be recouped or absorbed}}{\text{Efficiency of irrigation application}}$$

$$= \frac{4 \text{ cm}}{60\%} = \frac{4 \text{ cm}}{0.6} \text{ cm} = \mathbf{6.67 \text{ cm}}. \quad \textbf{Ans.}$$

Hence, 6.67 cm depth of irrigation water shall have to be applied to the fields at an interval of 16 days. **Ans.**

(c) (i) Explain how maximum flood discharge can be obtained from frequency analysis of streamflow data. (8)

(ii) The records of annual rainfall at a rain gauge station expressed in mm in chronological sequence from 1981 to 1992 are given below :

360, 285, 557, 815, 273, 229, 707, 478, 309, 176, 285, 651

Using Hazen's method, estimate the maximum annual rainfall which has a recurrence interval of 10 years. (12)

[Hint. Follow Solved Example 7.8 and use Hazen's probability equation (eq. 7.139) as $T = \frac{N}{m - 0.5}$; *i.e.*

$$10 = \frac{12}{m - 0.5}$$

$m = 1.7$; Therefore choose the 1st severest storm

$= \mathbf{815 \text{ mm}}$. **Ans.]**

(d) Discuss the salient features of Khosla's method of design of weir on pervious foundation, against sub-surface flow. Point out the merits and limitations of the method. (20)

Q. 10. *(a) Discuss the common causes of failure of tubewells. Indicate the possible remedial measures against each of these. What are the criteria of failure of tubewells?* (30)

(b) Two identical tube-wells penetrating fully a 12 m thick aquifer are located at 180 m apart. The tubewells have diameter of 30 cm, radius of influence of 300 m, and the coefficient of permeability of aquifer is 10^{-3} m/sec. Compute :

(i) Discharge of tube-well when only one is working with a drawdown of 5 m.

(ii) Percentage decrease in discharge of the well, if both are working with a drawdown of 5 m. (30)

[**Solution.** (*a*) Discharge of single tube-well is given by eqn. (16.20) as :

$$Q_1 = \frac{2\pi KHs}{2.3 \log_{10} \frac{R}{r_w}}$$

$$= \frac{2 \times 3.14 \times 10^{-3} \times 12 \times 5}{2.3 \log_{10} \frac{300}{0.15}} = \mathbf{0.0496\ m^3/s} \quad \textbf{Ans.}$$

(*b*) Discharge of each identical tube-well, when both are working, is given by eqn. (16.24) as :

$$Q_2 = \frac{2\pi KHs}{2.3 \log_{10} \left(\frac{R^2}{r_w \cdot B} \right)}$$

$$= \frac{2 \times 3.14 \times 10^{-3} \times 12 \times 5}{2.3 \log_{10} \left(\frac{300^2}{0.15 \times 180} \right)} = \mathbf{0.0465\ m^3/s.}$$

Percentage decrease in discharge

$$= \frac{0.0496 - 0.0465}{0.0496} \times 100 = \mathbf{6.3\%.} \quad \textbf{Ans.}]$$

Q. 11. *(a) What are the true regime conditions in an alluvial channel as stipulated by Lacey ?*

(ii) A stable channel is to be designed for a discharge of 40 m^3/sec and silt factor of unity. Calculate the dimensions of the channel using Lacey's regime equations. Also calculate the dimensions of the channel if it were to be designed on the basis of Kennedy's method with critical velocity ratio equal to unity, and the ratio of bed width to depth of flow the same as obtained from Lacey's method. (40)

[**Hint.** Follow *Solved Example 4.8*, and design the channel dimensions on Lacey's theory to be $y = \mathbf{1.84\ m}$, and $b = \mathbf{25.92\ m}$. **Ans.** Then, design channel dimensions on the basis of Kennedy's theory by using $\frac{B}{D}$ ratio as equal to

$\frac{25.92}{1.84} = 14.09$, and by following *Solved Example 4.6*, to compute $D = 1.53$ m and $B = 21.50$ m. **Ans.**]

(b) Write short notes on :

(i) Sprinkler irrigation *(ii) Drip irrigation*

(iii) Canal lining *(iv) Silt ejector.* (20)

Q. 12. *(a) Explain with sketches the various channel patterns of a river. Discuss the influence of these patterns on planning of a hydraulic structure.* (20)

(b) What are the objectives of river training ? Describe the following river training methods indicating their merits and demerits :

(i) Spurs *(ii) Guide banks*

(iii) Revetments. (20)

Year 1994 *(Civil Services)*

Q. 9. *(Compulsory) Answer any three sub divisions.*

(a) (i) How is the inconsistency of rainfall data of a rain gauge station ascertained ? Discuss briefly the technique and also enumerate the possible causes for such inconsistency.

(ii) What are monolayer films ? How effective are they in retarding evaporation from reservoirs. (20)

(b) (i) 800 m^3 of water is applied to a farmer's rice field of 0.6 hectares. When the moisture content in the soil falls to 40% of the available water between the field capacity (36%) of soil and permanent wilting point (15%) of the soil crop combination, determine the field application efficiency. The root zone depth of rice is 60 cm. Assume porosity = 0.4.

[**Note.** Please refer Solved Example 2.15].

(ii) Differentiate between "Potential Evapotranspiration" and "Actual Evapotranspiration". Do the characteristics of soil and available moisture content affect them ? If yes, how ? (20)

(c) (i) In what manner does seepage endanger the safety of a structure on permeable foundation ? How will you ensure the safety of structures against seepage effects.

(ii) Sketch the typical cross-section of channel, when it is (i) Fully in filling (ii) Partly in filling and partly in cutting. (20)

(d) (i) In a silt excluder for a barrage, why the tunnels in general are of different lengths ? Explain.

(ii) Give a typical layout of the Hydropower development "shortcircuiting long reaches of a circuitous river with steep slope".

(iii) Find the base width of elementary profile of a gravity dam from considerations of no tension. (20)

Q. 10. *(a) Compare the characteristics of Instantaneous unit Hydro-graph (IUH) with the characteristics of Conventional unit Hydrograph. Can S-curve hydrograph be used to derive an IUH of a catchment ? If so, how ?*

(b) The ordinates of a 6-hour unit hydrograph for a basin are given below :

Time (hours)	*0*	*3*	*6*	*9*	*12*	*18*	*24*
6-h unit hydrograph ordinates (m^3/s)	*0*	*150*	*250*	*450*	*600*	*800*	*700*
Time (hours)	*30*	*36*	*42*	*48*	*54*	*60*	*66*
6-h unit hydrograph ordinates (m^3/s)	*600*	*450*	*325*	*200*	*100*	*50*	*0*

A storm had three successive 6 hour interval of rainfall magnitudes of 3.0, 5.0 and 4.0 cm respectively. Assuming a ϕ-index of 0.20 cm/hr and a base flow of 30 m^3/s, determine the resulting hydrograph of flow. (40)

[**Hint.** Follow similar Solved Example 7.35].

Q. 11. *(a) What are "partially penetrating wells". Discuss their characteristics.* (10)

(b) A 30 cm well completely penetrates an unconfined aquifer of depth 40 m. After a long period of pumping at a steady rate of 1500 lpm, the drawdown in two observation wells 25 m and 75 m from the pumping well were found to be 3.5 m and 2.0 m, respectively. Determine the transmissibility of the aquifer. What is the drawdown at the pumping well ? Derive the equation used. (30)

[**Hint.** Derive and use Thiem's equation (*i.e.* eq. 16.9) for unconfined aquifers. Then to solve the given numerical example, assume the water table to be at the top of the aquifer, *i.e.* the depth of aquifer being equal to the saturated depth (d). Then using the eq. (16.9) we have

$$K = \frac{2.3\, Q \log \frac{r_2}{r_1}}{\pi (h_2^2 - h_1^2)} = \frac{2.3 \times \left(\frac{1.5}{60}\right) \text{m}^3/\text{s} \times \log_{10} \frac{75}{25}}{\pi [(40 - 3.5)^2 - (40 - 2)^2]}$$

$$= 0.781 \times 10^{-4} \text{ m/s}$$

$$T = K.d = 0.781 \times 10^{-4} \times 40$$

$$= \mathbf{3.126 \times 10^{-4}\ m^2/sec} \quad \textbf{Ans.}$$

Now, to determine drawdown, in main well s_w, use r_w in place of r_1, and h_w in place of h_1 in the above equation, as :

$$K = 0.781 \times 10^{-4} = \frac{2.3 \times 0.025\ \text{m}^3/\text{s} \times \log_{10} \frac{75}{0.15}}{\pi [36.5^2 - h_w^2]}$$

or $h_w = \mathbf{13.55\ m}$ **Ans.**]

(c) What are the different steps necessary to create an Engineering project ? Enumerating conservation purposes, differentiate between single purpose and multipurpose projects. (20)

[**Hint.** Please refer article 21.1 & 21.2 in "Hydrology and Water Resources Engineering — 1998 ed.]

Q. 12. *(a) Describe various modes of sediment transport in an alluvial channel.*

Design a Lacey's regime channel, flowing through material of average diameter 0.328 mm, for the following data :

Culturable commanded area = 1,00,000 hectare

Intensity of irrigation :

Rabi = 40%

Kharif = 30%

Outlet discharge factor :

Rabi = 1,800 hec/cumec

Kharif = 800 hec/cumec

Assume conveyance losses to be 10%. (30)

[**Hint.** Area irrigated in Rabi = 40,000 hec

$$\therefore \ Q \text{ for Rabi} = \frac{40{,}000 \text{ hec}}{1800 \text{ hec/cumec}} = 22.2 \text{ m}^3/\text{s}$$

$$\text{Similarly } Q \text{ for Kharif} = \frac{30{,}000 \text{ hec}}{800 \text{ hec/cumec}} = 37.5 \text{ m}^3/\text{s}$$

Hence, design the channel on Lacey's theory for Q = 37.5 m^3/s and silt factor $f = 1.75\sqrt{d_{mm}} = 1.75\sqrt{0.328} = 1.0$, by following the solved example 4.8, and compute $b = 25.06$ m, $y = 1.8$ m and $S = 1$ in 6110 for a channel having side slopes of $\frac{1}{2}H : IV$. **Ans.**].

(c) What are the objectives of river training ? Describe the following river training methods :

(i) Spurs *(ii) Guide banks.* (20)

Year 1995 (*Civil Services*)

Q. 9. *(Compulsory). Answer any three subdivisions :*

(a) In a basin six rain gauge stations are reporting rainfall. In a year the annual rainfall recorded by the gauges are as follows :

Station	A	B	C	D	E	F
Rainfall (cm)	41	51	32	55	50	68

For 8% error in estimation of the mean rainfall, calculate the optimum number of rain gauge stations in the basin. Give criteria to install them, if there number is more than the existing number. (20)

[**Hint.** Follow Example 7.3 and compute :

$$\bar{P} = \frac{41 + 51 + 32 + 55 + 50 + 68}{6} = 49.5 \text{ cm}$$

$$\overline{P^2} = \frac{\Sigma P^2}{n} = \frac{41^2 + 51^2 + 32^2 + 55^2 + 50^2 + 68^2}{6} = 2575.83$$

$$\sigma = \sqrt{\frac{n}{n-1}\left[\overline{p^2} - (\bar{p})^2\right]} = \sqrt{\frac{6}{5}\left[2575.83 - (49.5)^2\right]}$$

$$= \sqrt{150.7} = 12.276$$

$$C_v = \frac{100\,\sigma}{\bar{p}} = \frac{100 \times 12.276}{49.5} = 24.80$$

$$N = \left(\frac{C_v}{E}\right)^2 = \left(\frac{24.80}{8}\right)^2 = 9.61 \textbf{ Say 10 No.}$$

$\therefore$ Optimum No. of rain gauges = **10 No.** **Ans.**

4 additional gauges are, thus, required to be installed in the given catchment. *These additional gauges should be distributed in the different zones (caused by isohyets) in proportion to their areas.* This will ensure even distribution of the gauges over the entire basin. **Ans.**]

Q. 9. *(b) What are cross-drainage works ? The type of cross-drainage work on the main canal at a drainage can be changed by suitably changing the alignment of main canal. Explain giving schematic sketch.* (20)

Q. 9 *(c) What information is needed to route a flood through a reservoir? Discuss the salient features of the outflow hydrograph as it differs from the inflow hydrograph.* (20)

Q. 9 *(d) "Seismic forces are taken into account in the design of solid gravity dams". If the statement is correct, explain how ?* (20)

Q. 10 *(a) Briefly discuss the limitations of unit hydrograph theory.* (10)

Q. 10 *(b) The IUH (Instantaneous Unit Hydrograph) of a drainage basin is a triangle with base 36 hours and a peak of 20 m^3/s at 8 hours from the start. Determine 2 hour unit hydrograph* (20)

[Hint. Instantaneous Unit Hydrograph is a unit hydrograph (containing 1 cm effective rainfall) of infinitely small duration. It is only a theoretical concept, and is explained in author's another book titled "Hydrology and Water Resources Engg" under article 9.11.

The ordinates of a I.U.H. at D hr interval can be used to work out ordinates of unit hydrograph of D hr duration, by averaging the two adjoining coordinates. Say for example, mean of cordinates of IUH at 0 hr and D hr will represent the ordinate of U.H. at D hr. Similarly, the mean of the ordinates of IUH at D hr. and 2 D hr. will give the ordinate of the U.H at $2D$ hr. Hence, to work out ordinates of U.H. of 2 hr duration, the ordinates of IUH at 2 hr interval are first of all written down in col (2) of the solution table (Figs worked out from shape of IUH triangle).

Solution Table for Q. 10 (*b*)

Time (*1*)	*Ordinates of IUH at 2 h interval* (*2*)	*Ordinates of 2 h Unit hydrograph (Computed by averaging)* (*3*)	
0	0	0	
2	5	2.5	(a.v of 0 + 5)
4	10	7.5	(av. of 5 and 10)
6	15	12.5	(av. of 10 and 15)
8	20	17.5	(av. of 15 and 20)
10	18.57	**19.28**	
12	17.15	17.85	
14	15.73	16.43	
16	14.30	15.01	
18	12.87	13.58	
20	11.44	11.15	
22	10.00	10.72	
24	8.57	9.28	
26	7.14	7.85	
28	5.71	6.43	
30	4.28	5.00	
32	2.85	3.56	
34	1.43	2.14	
36	0	0	

The ordinates of unit graph are finally written down in Col (3) using averaging method clarified in the table **Ans.**]

Q. 10. *(c) The following flood hydrograph was recorded in a drainage basin due to a storm having three successive 6 hour interval of rainfall magnitudes of 2.9 cm, 4.9 cm, and 3.9 cm respectively. Assume base flow = 20 m^3/s, and storm loss = 0.15 cm/hour, determine a 6 hour unit hydrograph*

Time hours	*0*	*3*	*6*	*9*	*12*	*15*	*18*	*21*
Flood graph ord (m^3/s)	*20*	*50*	*92*	*140*	*199*	*202*	*204*	*144*
Time (hours)	*24*	*27*	*30*	*33*				
Flood graph ord (m^3/s)	*84.5*	*45.5*	*29.0*	*20.0*				

[**Hint.** This question is the reverse type of solved examples like 7.38 and can be solved by assuming ordinates of 6 hr unit graph as *a, b, c, d, e, f,* etc. and then computing the flood graph caused by three successive effective rainfalls amounting to (*i*) $2.9 - 0.15 \times 6 = 2$ cm: ; (*ii*) $4.9 - 0.15 \times 6 = 4$ cm ; and (*iii*) $3.9 - 0.15 \times 6 = 3$. cm, respectively. The second rainfall to occur at 6 hr lag, and third one at 12 hr lag. The additive hydrograph ordinates will equal the flood graph ordinates *minus* base flow. The computations are given below in the table.

Solution Table for Question 10 (*c*) of Year 1995

Time (hr)	*Assumed ordinates of U.H. of 6 hr m^3/s*	*Flood graph ordinate caused by effective of rainfall of 2.9 – 0.9 = 2 cm*	*Flood graph ordinate caused by effective rainfall = 4.9 – 0.9 = 4 cm at a lag of 6 hr*	*Flood graph ordinate caused by eff. rainfall = 3.9 – 0.9 = 3 cm at a lag of 12 hr*	*Σ Total flood graph (3) + (4) + (5) = Given value of flood graph ordinate — base flow of 20 m^3/s*
(1)	*(2)*	*(3)*	*(4)*	*(5)*	
0	0	0			$0 = 0$
3	a	$2a$			$2a = 30^*$; $a = 15$
6	b	$2b$	0		$2b = 72^{**}$; $b = 36$
9	c	$2c$	$4a$		$2c + 4e = 120$; $c = 30$
12	d	$2d$	$4b$	0	$2d + 4b = 179$; $d = 17.5$
15	e	$2e$	$4c$	$3a$	$2e + 4c + 3a = 182$; $e = 8.5$
18	f	$2f$	$4d$	$3b$	$2f + 4d + 36 = 184$; $f = 3$
21	g	$2g$	$4e$	$3c$	$2g + 4e + 3c = 124$; $g = 0$
24			$4f$	$3d$	$4f + 3d = 64.5$
27			$4g$	$3e$	$4g + 3e = 25.5$
30				$3f$	$3f = 9$
33				$3g$	$3g = 0$

* 50 – 20 = 30
** 92 – 20 = 72

The required ordinates of unit graph are thus finally worked out at 3 hr interval between 0 to 21 hrs, as :

0, 15, 36, 30, 17.5, 8.5, 3, 0 m^3/s. **Ans.**]

Q. 11. *(c) A 30 cm diameter well completely penetrates a confined aquifer of permeability 45 m/day. The length of the strainer is 20 m. Under steady state of pumping, the drawdown at the well was found to be 3.0 m, and the radius of influence was 300 m. Calculate the discharge. With increase in diameter of well by 50%, what will be the increase in % of discharge ? Derive the equation used* (30)

[**Hint.** For confined stratum, eqn. (16.20) is applicable, which states :

$$K = \frac{2.3\, Q \log_{10} \frac{R}{r_w}}{2\pi\, H.S}$$

where $K = 45$ m/day $= \dfrac{45}{24 \times 3600}$ m/s

$= 0.052 \times 10^{-2}$ m/s

$R = 300$ m

$r_w = 0.15$ m

$H = 20$ m

$s = 3.0$ m

$$\therefore \quad Q = \frac{2\cdot\pi H.S.K}{2.3\log_{10}\frac{R}{r_w}} = \frac{2\times 3.14\times 20\times 3\times 0.052\times 10^{-2}}{2.3\times\log_{10}\frac{300}{0.15}}$$

$$= 0.02581\ \text{m}^3/\text{sec} = 25.81\ \text{L/sec} \quad \textbf{Ans.}$$

If well dia increases by 50%, then r_w will become = 1.5 r_w ; then

$$Q' = \frac{2\pi H.S.K}{2.3\log\frac{R}{1.5\,r_w}}$$

$$\therefore \quad \frac{Q'}{Q} = \frac{\log_{10}\frac{R}{r_w}}{\log_{10}\frac{R}{1.5\,r_w}}$$

$$\%\text{ age increase in discharge} = \frac{Q'-Q}{Q}\times 100 = \left(\frac{Q'}{Q}-1\right)100$$

$$= \left[\frac{\log_{10}\frac{300}{0.15}}{\log_{10}\frac{300}{0.15\times 1.5}} - 1\right]100 = \left[\frac{\log_{10}2000}{\log_{10}\frac{2000}{1.5}} - 1\right]100$$

$$= \left[\frac{3.3010}{3.1249} - 1\right]100 = \textbf{17.61\%} \quad \textbf{Ans.}]$$

Q. 11. *(b) Write short explanatory notes on :*

(i) Flood frequency analysis

(ii) Guide bunds and Launching aprons (30)

Q. 12. *(a) Design a cement plaster lined irrigation channel (n = 0.012) for 30 m^3/s discharge at a slope of 22.5 cm/km. Take the side slope as $1\frac{1}{2}$ H : 1 V.*

[Hint. This question can be solved by following the solved example 5.4, and by assuming a limiting velocity of say 1.5 m/sec in the channel]

Q. 12. *(b) Calculate the seepage through an earth dam resting on an impervious foundation for the relevant data given below*

Height of the dam	*= 60.0 m*
Upstream slope	*= 2.75 : 1 (H : V)*
Downstream slope	*= 2.50 : 1 (H : V)*
Free board	*= 2.5 m*
Crest width	*= 8.0 m*
Length of drainage blanket	*= 120.0 m*

Coefficient of permeability of the embankment material

$$X\text{- direction} = 4.0 \times 10^{-7} \text{ m/s}$$

$$Y\text{-direction} = 1 \times 10^{-7} \text{ m/s} \quad (20)$$

[Hint. Exactly similar question has appeared in Engg services 1995 Exam, which has been solved on page there itself. Students may follow through, and compute

$$q = 35.24 \times 10^{-7} \text{ m}^3/\text{sec. m}$$

$$= 305 \text{ l per day per m width of dam} \quad \textbf{Ans.}$$

Q. 12. *(c) Why is it necessary to control sediment entering a canal ? Describe the working of a silt ejector giving neat sketches. On what basis is it designed ?* (20)

Year 1996 (*Civil Service*)

Q. 9. *(Compulsory) Answer any three sub divisions.*

(a) What are the causes of infertility of water-logged areas ? Derive an expression to determine the spacing of drains in open drainage system. Under what conditions this system may be recommended as an antiwater logging measure. (20)

(b) Sketch a 'runoff cycle'. Discuss direct runoff and base flow components of the cycle in details (20)

(c) Find the base width of the elementary profile of a gravity dam from the considerations of no tension, and safety in sliding. Assume reservoir full condition. (20)

(d) Define critical shear stress for a sediment bed. Discuss the use of Shield's diagram to calculate the value of critical shear stress if sediment and fluid properties are known. (20)

Q. 10. *(a) Define a unit hydrograph. Enumerate steps to develop a unit hydrograph from a given flood hydrograph for a catchment. The peak of a flood hydrograph due to a 6 hour storm is 470 m^3/s. The average depth of rainfall is 8.0 cm. Assume an infiltration loss of 0.25 cm/hour and a constant base flow of 15 m^3/s. Estimate the peak discharge of a 6 hour unit hydrograph for the catchment.* (35)

[Hint. For subjective part, please refer text. For numerical question, the effective rainfall is = $8.0 - 6 \times 0.25 = 6.5$ cm. The U.H. will be obtained by dividing the ordinates a flood graph by 6.5. Hence, the peak of U.H.

$$= \frac{\text{Peak of F. graph} - \text{Base flow}}{6.5}$$

$$= \frac{470 - 15}{6.5} = 70 \text{ m}^3/\text{s}$$

Add constant base flow of 15 m^3/s

$\therefore$ Peak discharge of U.H. of 6 hr duration

$$= 70 + 15 = \mathbf{85 \text{ m}^3/\text{s}} \quad \textbf{Ans.}]$$

Q. 10. *(b) How would you determine a 2 hour synthetic unit hydrograph by Snyder's method ? Discuss the situations in which this method is used* (25)

[Hint. Please refer article 9.9 in author's another book titled "Hydrology and Water Resources Engineering"]

Q. 11. *(a) In a frequency analysis of rainfall based on years of data of 10 minutes storm, the following values were obtained :*

Arithmetic mean of data = 1.65 cm

Standard deviation = 0.45 cm

Find using Gumbel's extreme value distribution the recurrence interval of a storm of ten minute duration and depth equal to 3 cm. (20)

[Hint. Use Eq. (7.142) as

$$X_{(T)} = \bar{X} + K \cdot \sigma$$

or $$3 = 1.65 + K \times 0.45$$

$$K = 3$$

Use eq. (7.148) as

$$K = \frac{y_{(T)} - 0.577}{1.2825} \text{ for } N \to \alpha$$

Since the analysed data is for a number of years, it can be presumed to be for an infinite no. of yrs ; *i.e.* $N \to \alpha$

Hence, $$3 = \frac{Y_{(T)} - 0.577}{1.2825}$$

$$Y_{(T)} = 4.4245$$

Finally use eq. (7.146) as

$$Y_{(T)} = -\left[\ln \ln \frac{T}{T-1}\right] \quad \text{or} \quad -4.4245 = \ln \ln \frac{T}{T-1}$$

or $$\frac{T}{T-1} = 1.012052 \quad \text{or} \quad \frac{T-1}{T} = 0.98809$$

or $$T = 83.97 \text{ ; say } \mathbf{84 \text{ yrs Ans}}]$$

Q. 11. *(b) Draw a schematic sketch of a diversion headworks with canals taking off from both sides. Indicate the functions of undersluices portion and canal head regulator.* (25)

Q. 11. *(c) Discuss the use of hydraulic jump, as an energy dissipation device in the design of hydraulic structures* (15)

Q. 12. *(a) Explain the terms :*

(i) Specific yield *(ii) Specific capacity*

A well penetrates into an unconfined aquifer having a saturated depth of 100 m. The discharge is 250 litres per minute at 12 m draw down. Assuming equilibrium flow conditions and a homogeneous aquifer, estimate the discharge at 18 m drawdown. The radius of influence may be taken as equal in both cases. Derive the formula used (30)

[Hint. Please refer solved example 16.3. **This question was set from this book]**

Q. 12. *(b) Write short notes on :*

(i) Piping failure of a weir floor and remedial measures

(ii) Spherical flow in a well (30)

Year 1997 *(Civil Services)*

Q. 9. *(Compulsory). Answer any three sub divisions*

(a) Explain briefly interception, transpiration and consumptive use. A 3-hour storm occurs over a 63 sq. km area. From the following data, estimate the rainfall excess for the entire area and its hourly distribution :

Sub area	*ϕ Index*	*Hourly rain*		
	cm/hr	*1st*	*2nd*	*3rd*
14.0	2.0	1.5	5.0	0.8
21.0	3.5	1.5	5.0	0.9
28.0	1.0	1.7	6.0	1.5

(20)

[Hint. Follow similar solved Example 7.28, and determine.

Total rainfall excess (runoff) = Σ (partial runoff volumes)

or $R = R_1 + R_2 + R_3$

$R_1 = 14 \text{ km}^2 [0 + (5.0 - 2.0) + 0] = 42 \text{ km}^2 \cdot \text{cm}$

$R_2 = 21 \text{ km}^2 [0 + (5.0 - 3.5) + 0] = 31.5 \text{ km}^2 \cdot \text{cm}$

$R_3 = 28 \text{ km}^2 [(1.7 - 1.0) + (6.0 - 1.0) + (1.5 - 1.0)]$

$= 28 \text{ km}^2 [0.7 + 5 + 0.5] = 173.6 \text{ km}^2 \cdot \text{cm}$

$\therefore \quad R = R_1 + R_2 + R_3 = 42 + 31.5 + 173.6 = 247.1 \text{ km}^2 \cdot \text{cm}$

$\therefore$ Total runoff as volume from 63 km^2 area = 247.1 $\text{km}^2 \cdot \text{cm}$

$\therefore$ Total runoff as depth $= \dfrac{247.1}{63} \text{ cm} = \mathbf{3.92 \text{ cm}}$ **Ans.**

Hourly values of total runoff can also similarly be computed by considering hourly values separately rather than in consolidated form as below :

$$\text{Total runoff in 1st hour} = \frac{14 \times 0 + 21 \times 0 + 28\,(1.7 - 1.0)}{63}$$

$$= \mathbf{0.31 \text{ cm}} \qquad ...(i)$$

$$\text{Total runoff in 2nd hour} = \frac{14 \times (5.0 - 2.0) + 21\,(5.0 - 3.5) + 28 \times (6.0 - 1.0)}{63}$$

$$= \frac{14 \times 3 + 21 \times 1.5 + 28 \times 5.0}{63}$$

$$= \mathbf{3.39 \text{ cm}} \qquad ...(ii)$$

$$\text{Total runoff in 3rd hour} = \frac{14 \times 0 + 21 \times 0 + 28\,(1.5 - 1.0)}{63}$$

$$= \frac{28 \times 0.5}{63} = \mathbf{0.22 \text{ cm}} \qquad ...(iii)$$

Total runoff = (*i*) + (*ii*) + (*iii*)

= 0.31 + 3.39 + 0.22 = **3.92 cm** (*O.K*) **Ans.**

(b) Write a note on the main elements of water resources development. Enumerate the steps followed in planning. (20)

(c) Explain the various causes of water logging. Describe various methods adopted as anti-water logging measures. (20)

(d) Describe in brief various types of groynes used for river training. Draw a section of a groyne. (20)

Q. 10. *(a) What do you understand by infiltration index ? How do you determine it?* (15)

(b) Describe with the help of a neat sketch any three methods of separation of base flow from the hydrograph of runoff indicating the situations under which you advocate them. (20)

(c) The following are the ordinates of a 3 hour unit hydrograph. Derive the ordinates of a 6 hour unit hydrograph and plot the same.

Time (hr.)	*3 hr. UGO (cumec)*	*Time (hr.)*	*3 hr. UGO (cumec)*
0	*0*	*15*	*9.4*
3	*1.5*	*18*	*4.6*
6	*4.5*	*21*	*2.3*
9	*8.6*	*24*	*0.8*
12	*12.0*	*27*	*0.0*

(25)

[**Hint.** Follow exactly similar solved Example 7.33]

Q. 11. *(a) A 30 cm well penetrates 50 m below the static watertable. After a long period of pumping at a rate of 1800 lpm, the draw-downs in the wells at 15 m and 45 m from the pumped well were 1.7 m and 0.8 m respectively. Find the transmissibility of the aquifer. What is the draw-down in the pumped well ?* (30)

[**Hint.** Follow exactly similar solved Example 16.5]

(b) Derive an expression for discharge from a well fully penetrating a confined aquifer. (15)

(c) What do you understand by 'priming' and 'depriming' ? Describe any three devices used for early priming in Saddle Siphon Spillway. (15)

Q. 12. *(a) What is berm ? Why is it provided in canals ? Using Lacey's theory, design an irrigation channel for the following data :* (30)

Discharge Q = 50 cumec

Silt factor f = 1

[**Hint.** Follow solved Example 4.8]

Q. 12. *(b) Write short notes on :*

(i) Bed forms in alluvial channels

(ii) Silt excludes

(iii) Divide wall and fish ladder. (30)

Year 1998 (*Civil Services*)

Q. 9. *(Compulsory). Answer any three sub divisions :*

(a) Rainfall intensities in mm/h at half-hour intervals during a 4-hr storm were : 5,

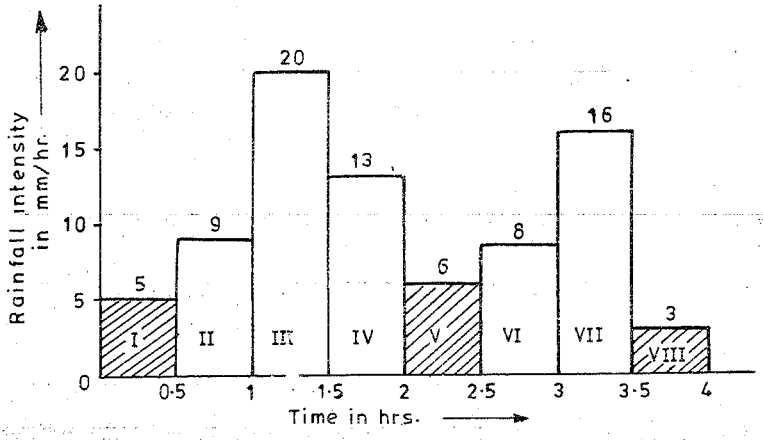


Fig. 31.12

9, 20, 13, 6, 8, 16 and 3 mm/hr. If the corresponding observed runoff is 27.45 million m^3 from a basin having an area of 1830 km^2, determine the ϕ_{index} for the storm. (20)

[Solution. Follow Solved Example 7.26, and compute as follows : Plot rainfall hyetograph as shown below.

$$\text{Total runoff } (Q) = \frac{27.45 \times 10^6 \text{ m}^3}{(1830 \times 10^6) \text{ m}^2} = \frac{27.45}{1830} \times 1000 \text{ mm} = \mathbf{15 \text{ mm}}$$

$$\text{Total rainfall}(P) = 0.5 \text{ hr } [5 + 9 + 20 + 13 + 6 + 8 + 16 + 3] \frac{\text{mm}}{\text{hr}}$$

$$= \mathbf{40 \text{ mm}}$$

$$W_{index} = \frac{P - Q}{t_r} = \frac{(40 - 15) \text{ mm}}{4 \text{ hr}} = \mathbf{6.25 \text{ mm/hr}}$$

Since ϕ_{index} has to be somewhat more than W_{index}, we can conclude that ϕ_{index} would be a little more than 6.25 mm/hr. When this is so, evidently, the Ist, Vth and VIIIth (half hr) rainfalls would become ineffective in producing excess rain, because the rain intensity in those periods would be less than ϕ_{index}.

In other words, the period (t_e) during which excess rain occurs, would be only $4 - 1.5 = 2.5$ hr. In that case, using Eq. (7.102), we have

$$\phi_{index} = \frac{\text{Total in filtration during the period of excess rainfall}}{\text{Period of excess rainfall}}$$

$$= \frac{\text{Total infiltration} - \text{Infiltration during the time when no excess rain occurs}}{t_e}$$

$$= \frac{25 \text{ mm} - 0.5 [5 + 6 + 3]}{2.5 \text{ hr}}$$

$$\therefore \phi_{index} = \frac{25 - 0.5 \times 14}{2.5} = \frac{25 - 7}{2.5} = \frac{18}{2.5} = 7.2 \text{ mm/hr.}$$

With this computed value of ϕ_{index}, no other period excepts the periods marked as II, VI & VIII (already accounted) become infructous to yield excess rain. Hence, the computations done do not need further revision. Hence, $\phi_{index} = \mathbf{7.2\ mm/hr}$ **Ans.**

Q. 9. *(b) Distinguish between*

(i) Drainage divide and ground water divide

(ii) Consumptive and non-consumptive uses of water (20)

Q. 9. *(c) Briefly explain water-logging and its ill-effects.* (20)

Q. 9. *(d) Draw a sketch of a composite earth dam showing its components. Also state the functions of these components.* (20)

Q. 10. *(a) During a recuperation test, the water level in an open well depressed by 2.5 m during pumping and it recuperated 2 m in 100 minutes. Find :*

(i) Yield from a well of 4 m diameter under a depression head of 2.4 m; and

(ii) Diameter of a well to yield 12 l/s under a depression head of 2.4 m. (20)

[**Hint.** Follow exactly similar Solved Example 16.10]

Q. 10. *(b) Describe the zone method of design of high gravity dams.* (20)

Q. 10. *(c) What is a silt excluder ? Explain its working with the help of neat sketches.* (20)

Q. 11. *(a) The ordinates of a 3-hr unit hydrograph are as follows :*

Time (hr.)	*3-hr unit graph ordinates (m^3/s)*
0	0
3	3
6	8
9	15
12	13
15	10
18	6
21	2
24	0

Assuming a constant base flow of 15 m^3/s and an average storm loss of 0.20 cm/hr, obtain the ordinates of the resultant hydrograph for three successive storms of 3.6, 6.0 and 4.2 cm, each of 3 hours duration. (20)

[**Hint.** Follow exactly similar Solved Example 7.38]

(b) Describe various methods used for energy dissipation below spillways. (20)

(c) Compare Kennedy's and Lacey's silt theories. Which of them would you prefer for designing an irrigation channel ? Why ? (20)

Q. 12. *(a) Explain the terms 'duty' and 'delta'. State the different factors that affect duty.* (20)

(b) Draw a neat sketch of a syphon aqueduct and state the conditions under which it is recommended. (20)

Q. 12. *(c) Write short notes on :*

(i) Reservoir sedimentation ; and (ii) River training. (20)

Year 1999 (*Civil Services*)

Questions 9 is compulsory

Q. 9. Answer any three sub-divisions

(*a*) *Define a flood plain. Briefly describe and discuss structural and non-structural measures of flood control.* (20)

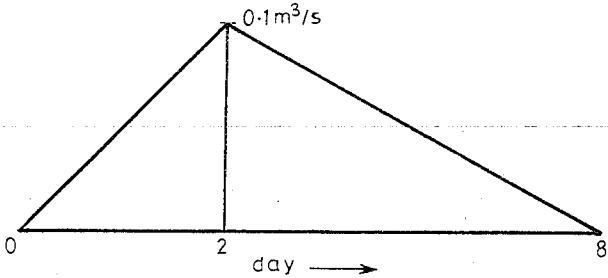


Fig. 31.13

(*b*) *Define a unit hydrograph. What are its assumptions ?*

The unit hydrograph of a drainage basin for unit duration of 1 hour and effective rainfall of 1 cm/hour is given in the form of a triangle with the base as 8 days and peak flow of 0.1 m^3/sec occurring on the second day. Estimate the area of the drainage basin.

[**Solution.** Area of unit hydrograph = 1 cm of rain on catchment of A sq. m. .

$$\text{Area of U. graph} = \frac{1}{2} \times 0.1 \frac{\text{m}^3}{\text{s}} \times 8 \text{ days}$$

$$= \frac{1}{2} \times 0.1 \frac{\text{m}^3}{\text{s}} \times (8 \times 24 \times 60 \times 60) \text{ . s}$$

$$= \frac{1}{2} \times 0.1 \times 8 \times 24 \times 3600 \text{ m}^3$$

or $$\frac{1}{2} \times 0.1 \times 8 \times 24 \times 3600 \text{ m}^3 = \left(\frac{1}{100} \text{ m} \right) \times A \text{ (m}^2\text{)}$$

or $$A = 345.6 \times 10^4 \text{ m}^2 = 345.6 \text{ ha}$$ **Ans.**]

(*c*) *What are reservoir routing and channel routing ? What are the shortcomings of the Muskingum method of flood routing ?* (20)

[**Hint :** Please refer chapter on Flood Forecasting in "Hydrology and water Resources" by the same author.]

(*d*) *What are the important features for the design of cross-drainage works ? Sketch a typical siphon aqueduct.* (20)

Q. 10. (*a*) (*i*) *State and discuss the various factors affecting run off from a drainage basin.*

(*ii*) *How will you test the consistency of the rainfall record of a station, if the annual precipitation data of a group of stations including the one in question are available for a long period ?* (20)

Q. 10. (*b*) *At a certain point in an unconfined aquifer of 4 km^2 area, the water-table was at an elevation of 120.0 m. Due to natural recharge in a wet season, its level rose to 121.2 m A volume of 2 million cubic metres of water was then pumped out of the aquifer, causing the water-table to reach a level of 119.2 m. Assuming the water-table in the entire aquifer to correspond in a similar way, estimate*

(*i*) *the specific yield of the aquifer; and*

(*ii*) *volume of recharge during the wet season.* (20)

[**Solution.** Water pumped in lowering W.T. by 121.2 – 119.2 = 2 m is 2M.m^3.

Vol of Aquifer drained in lowering W.T. by 2m

$$= \text{Area} \times 2 \text{ m depth} = (4 \times 10^6 \text{ m}^2) \times 2 \text{ m} = 8 \text{ M.m}^3$$

(*i*) Sp. yield of the aquifer (Eq. 16.8)

$$= \frac{\text{Vol. of water drained by gravity drainage}}{\text{Total vol. of aquifer drained}}$$

$$= \frac{2 \text{ M.m}^3}{8 \text{ M.m}^3} = 0.25 = 25\% \quad \textbf{Ans.}$$

(*ii*) During rainy season, the W.T. rose by 121.2 – 120 = 1.2 m. Since 2m lowering of W.T. equals 2 M.m^3 of water withdrawal, 1.2 m rise will be equal to **1.2 M.m^3 of recharge. Ans.**]

Note. This question was earlier set at AMIE Exam., 1994 (Summer) and its solution was available in Author's another book titled "Hydrology and Water Resources Engg."

Q. 10. (*c*) *Briefly discuss different methods of economic evaluation of water resources projects. What is meant by project-induced economic growth ?* (20)

Q. 11. (*a*) *State the method of images as used in the analysis of ground-water flow.*

A 0.5 m diameter well fully penetrates a 30 m thick unconfined aquifer of hydraulic conductivity 20 m/day. Due to continuous pumping, if a drawdown of 1.0 m is registered in the well, what is the rate of pumping when the well is located at a distance of 50 m from a perennial stream ?

[**Hint.** Pl refer solved example 16.9] (30)

Q. 11. (*b*) *What are the factors affecting the evaporation process ? Briefly describe the Penman approach of estimating reservoir evaporation.* (15)

Q. 11. (*c*) *What is the purpose of providing drainage in an irrigated system ? What are the effects of poor drainage on soil and plants.* (15)

Q. 12. (*a*) *Enumerate the various forces acting on a gravity dam. Also indicate their points of application. Why are construction joints provided in a dam ?* (20)

Q. 12. (*b*) *Briefly describe and discuss the various methods of lining irrigation canals. Give a cross-section of a lined canal.* (20)

Q. 12. (*c*) *Suggest a suitable device for energy dissipation at the toe of a spillway with the following data :*

Discharge per unit width	= *10 cumecs.*
Depth of water at the toe	= *80 cm*
Margin of safety to be adopted against sweep out	= *7%*
Tail water depth	= *6.0 m*

(20)

[**Hint.** Here $y_1 = 0.8$ m ; T.W.D. = 6 m ; $q = 10\ m^3/s$

Using Eq. (10.4) as :

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}}, \text{ we get}$$

$$y_2 = -\frac{0.8}{2} + \sqrt{\frac{0.8^2}{4} + \frac{2 \times 100}{9.81 \times 0.8}} = 4.664 \text{ m.}$$

With 7% margin for sweep out caused by jumping water on tail water, y_2 becomes equal to $1.07 \times 4.664 = 4.99$ m.

Since tail water depth = 6m, is much more than $y_2 = 4.99$ m, the jump to be formed at the toe of the spillway shall be drowned out by tail water, and proper energy dissipation will not occur.

Hence, the point of jump formation will have to be raised by providing a *sloping glacis* as shown in fig. 21.31 (b_1) - allowing the jump to form on the apron at RL above the river bed by as much as TWD–$y_2 = 6 - 4.99 = 1.01$ m. Alternatively a *roller bucket* as shown in Fig. 21.31 (b_2) may be provided for energy dissipation. **Ans.**]

Year 2000 (*Civil Services*)

(Max. 3 Questions can be solved from this section)

Section B

Q. 5. *is compulsory in five parts :*

Q. 5(*a*). *Briefly discuss the factors affecting run off from a drainage basin* (12)

Q. 5(*b*). *Describe the chemical constituents which affect the suitability of water for irrigation.* (12)

Q. 5(*c*). *How will you estimate the benefits of*

(*i*) *on irrigation project ; and* (*ii*) *a flood control project* (12)

Q. 6.(*a*). *Define storage coefficient and transmissibility coefficient of an aquifer. A well is 30 cm in diameter and fully penetrates 50 m below the static water-table. After 36 hours of pumping at a steady state rate of 4 m^3/minute the water level in a test well 200 m apart is lowered by 1.2 m; and in a well 40 m away, the draw down is 2.7 m. What is the transmissibility of the aquifer ?* (20)

[**Hint.** Pl. follow exactly similar Solved Question 16.5.]

Q. 6(*c*). *Briefly discuss the causes of water logging in irrigated areas. Describe the procedure for the design of the drainage for water-logged soils.* (20)

Q. 7(*a*). *Describe methods of measuring rainfall and snowfall of a basin. How is consistency of rainfall data determined.* (20)

Q. 7(*c*). *Describe salient features of meandering of rivers in alluvial plains. Describe suitable methods of training such rivers.* (20)

Q. 8(*a*). *Briefly describe Lacey's theory of transportation of silt in channels*

Design an irrigation channel in alluvial soil according to Lacey's Regime theory. The following data are given :

Full supply discharge = 15 cumecs *Lacey's silt factor = 1.0 ;*

Channel side slopes = 1/2 : 1. (20)

[**Hint.** Pl refer exactly *Similar Solved Example* 4.9]

Q. 8(*c*). *Briefly discuss various methods of determining flood frequency analyses. Develop a computer program to determine flood frequency by any one method.* (20)

Year 2001 (*Civil Services*)

Section B. (*Max. 3 questions can be solved from this section*)

Q. 5. *is compulsory in five parts from Jrr. & Env. Engg. side.*

Q. 5 (*a*). *What is the difference between a 'confined aquifer' and an 'unconfined aquifer' ? Define and describe Darcy's law of flow of water through permeable soils.* (12)

Q. 5 (*b*). *What is the water requirements of crops ? What is the meaning of the terms 'Duty' and 'Delta' in connection with crop water requirements ? What are the factors affecting duty ? What is average 'delta value' in millimetres for sugarcane ?* (12)

Q. 5 (*c*). *What are the various types of cross drainage works ? What is the purpose of these C. D. Works ? Describe the use of syphon in cross drainage works.* (12)

Q. 6 (*a*). *What are the various surface water sources used for irrigation ? How is the storage capacity of a large reservoir fed by a river for a large irrigation project determined ?* (20)

Q. 6 (*b*). *For reducing seepage of water through the body of an earthen dam, what provisions are made ? What is piping in earthen dams ? How is it prevented ?* (20)

Q. 6 (*c*). *Discuss hydrological cycle and water budget. How can we determine yield from a catchment for various uses from natural rainfall ?* (20)

Year 2002 (Civil Services - Paper II)

Section - B. (*Maximum of 3 Questions can be solved from this section as against a total of 5 Questions).*

Q5 is compulsory in five parts from this section from Irrigation and Environment Engg side.

Q5. (*a*) *List the different types of canal lining in common use. Draw a neat sketch of a typical cross-section of a canal carrying a discharge of 60 m^3/s and lined with brick in cement mortar. Mark the salient features on the sketch.* (12)

Q5. (*b*) (*i*) *What are the basic premises of unit hydrograph theory ? What are the limitations of unit hydrographs ?*

(*ii*) *A 6-hour unit hydrograph is triangular in shape and has a peak ordinate of 10 m^3/s. The base is 72 hours with 12 hours of time up to the peak. What is the area of the catchment represented by this unit hydrograph.* (12)

[**Hint :** Vol. of water contained in $U \cdot H$

$$(1\text{ cm})\,A\,(m^2) = \frac{1}{2} \times 72\ hrs \times 10\,\frac{m^3}{s} \qquad \text{where } A \text{ is catchment area in } m^2.$$

or $$\left(\frac{1}{100}\text{ m}\right) A\,(m^2) = \frac{1}{2} \times (72 \times 3600)\ s \times 10\,\frac{m^3}{s}$$

or $$A = (36 \times 36000 \times 100)\ m^2 = 12960 \times 10^4\ m^2 = 12960 \text{ hectares} \quad \text{Ans.}]$$

Q5. (*c*) *In Khosla's theory, how is the exit gradient of a weir in permeable foundation estimated ? Using the theory, how can the factor of safety of a weir design against piping be estimated ?* (12)

[**Hint :** Exit gradient at downstream end of a weir having depth of *d/s* cut off $= d$ is given by equation (11.4) as :

$$G_E = \frac{H}{d}\,\frac{1}{\pi\sqrt{\lambda}}$$

where $$\lambda = \frac{1 + \sqrt{1 + \alpha^2}}{2} \text{ and } \alpha = \frac{b}{d}\ ; \qquad \text{where } b = \text{Total floor length}$$

$$F \cdot S = \frac{1}{\text{Exit gradient}}.$$

Q5. (*f*) *Write the Shield's curve for incipient motion of sediment. Mark clearly the ordinate and abscissa of this curve and level the salient features.*

[**Hint :** Please refer article 4.5 and Fig. 4.4]

Q6. (*b*) *In the context of sediment transport in alluvial channels, describe briefly the following :*

(*i*) *Bed load,* (*ii*) *Bed forms and* (*iii*) *Regime channel* (20)

Q6. (*c*) *List the different methods of application of irrigation water to farm land. In a system which relies on supply of water through field channels, indicate methods of achieving high efficiency of water use.* (20)

Q7. (*a*) (*i*) *Write the Rouse's equation for distribution of suspended sediment in the vertical of a channel and label all the terms.*

In a wide alluvial channel having suspended sediment load, the depth of flow is 2.8 m. If a suspended load sampler indicated a concentration of 700 ppm at a point 30 cm below water surface, estimate the concentration at a point 10 cm above the bed in the same vertical. The exponent in Rouse's equation can be assumed as 0.4. (20)

[**Solution.** Rouse's equation is given by equation (4.40) as :

$$\frac{c}{c_a} = \left[\frac{a\,(D - y}{y\,(D - a)}\right]^{\frac{w_o}{KV*}} \qquad \text{...(Eqn. 4.40)}$$

In the given question, the exponent $\frac{w_o}{KV*}$ is given as equal to 0.4.

Hence, the equation becomes :

$$\frac{c}{c_a} = \left[\frac{a\,(D - y}{y\,(D - a)}\right]^{0.4}$$

where c_a is known concentration at a distance 'a' from the bed c is concentration at distance y from load.

Here $D = 2.8$ m.

$c_a = 700$ ppm, $a = (2.8 - 0.3) = 2.5$ m

$c = ?$ $y = 0.1$ m

$$\therefore \quad \frac{c}{700 \text{ ppm}} = \left[\frac{2.5\,(2.8-0.1)}{0.1\,(2.8-2.5)}\right]^{0.4} = \left[\frac{2.5 \times 2.7}{0.1 \times 0.3}\right]^{0.4} = (225)^{0.4} = 8.727$$

or $c = 6109$ ppm Ans.

Q7. (*c*) *Explain the difference between hydraulic routing and hydrological routing of a flood. Describe method of routing of flood wave through a reservoir with uncontrolled spillway outlet ?* (20)

[**Hint :** Please see article 18.10].

Q8. (*a*) *Describe the principle used in the development of Ogee spillway profile. Describe the discharge characteristics of Ogee spillways with vertical upstream face under free flow condition.* (20)

[**Hint :** Please see articles 21.6.2 & 21.6.4 to 21.6.6.]

Q8. (*c*) *Discuss the problem of reservoir sedimentation ? What is the impact of sedimentation on the performance of a multipurpose reservoir ? What remedial measures can be adopted to reduce the rate of siltation ?* (20)

[**Hint :** Please see article 18.12 & 18.12.5.]

Year 2003 (*Civil Services Paper II*)

Q5. (*a*) **Compulsory :** *Briefly state the various steps needed for planning an irrigation project.* (12)

Q5. (*b*) *List the objectives of various of water resources development in the context of the lesser developed countries.* (12)

Q5. (*c*) *Briefly describe the basic methods of applying irrigation water to fields describing the situations when each method is useful.* (12)

Q6. (*a*) *In a lake of 70.8 ha surface, the inflow was 1.5 m^3/s in a certain 30 day month. The dam on the lake regulated the outflow (discharge) from the lake to 1.25 m^3/s in that month. If the recorded precipitation in that month was 7.62 cm and the storage volume increased by an estimated 6,50,000 m^3, what is the estimated evaporation in m^3 and cm ? Assume that no water infiltrates out of the bottom (and sides) of that lake.* (20)

[**Hint.** Inflow = 1.5 m^3/s ; Outflow = 1.25 m^3/s

Increase in storage (rate) = 0.25 m^3/s

Increase in storage in the month of 30 days

$$= 0.25 \times (30 \times 24 \times 3600)\ m^3 = \mathbf{6,48,000\ m^3}.$$

Precipitation = 7.62 cm

Precipitation volume = 7.62 cm × Lake area of 70.8 ha

$$= \frac{7.62}{100}\ m \times (70.8 \times 10^4\ m^2) = \mathbf{53,950\ m^3}.$$

Total gross storage $= 6,48,000 + 53,950 = 7,01,950\ m^3$.

Actual net increase in storage = 6,50,000 cum

Total loss of water volume from the lake

$$= 7{,}01{,}950 - 6{,}50{,}000 = 51{,}950 \text{ cum}$$

Since loss due to infiltration is Nil, the total loss is the loss due to evaporation

= 51,950 cum. Ans.]

Q7. (*a*) *Assuming the initial infiltration rate of 10 mm/h and final infiltration rate of 5 mm/h and the constant value (describing the rate of decay of the difference between the initial and final infiltration rates) as 0.95 h^{-1}, calculate the total infiltration depth for a storm lasting 6 h.* (20)

[**Hint.** Please refer Solved Example 7.13 (*a*).]

Q8. (*a*) (*iii*) *Briefly state the criteria for selection of gravity dam site.* (10)

Year 2004 (*Civil Services Paper II*)

Q5. (*a*) *Why are canal falls provided in irrigation channels ? With the help of sketches, describe and illustrate a trapezoidal notch fall, labelling all the salient features and their functions in flow management and safety.* (12)

(*b*) *What is meant by river training ? What are its objectives ? What are the in-situ features that help in this context ?* (12)

(*c*) *Estimate the depth and frequency of irrigation required for a certain crop, given :*

Root zone depth	:	*90 cm*
Field capacity	:	*22%*
Wilting point	:	*12%*
Apparent sp. gr. of soil	:	*1.5*
Consumptive use	:	*22 mm/day*
Efficiency of irrigation	:	*60%*

Assume 50% depletion of available moisture as the indiator to begin application of irrigation water.

[**Solution.** Please follow similar solved example 2.14 and compute as :

Available moisture = Field capacity m.c. – Wilting point m.c.

$$= 22\% - 12\% = 10\%$$

Since irrigation water is to be supplied on 50% depletion of available moisture, we have

Readily available moisture = 50% × 10% = 5%

It means that irrigation water will be supplied as soon as m.c. falls to 17% from 22%. Thus, m.c. will be allowed to vary between 22% to 17%.

Depth of irrigation water required to raise m.c. from 17% to 22%

$$= \frac{\gamma_d \cdot d}{\gamma_w}\left[\text{Field capacity m.c.} - \text{Optimum m.c.}\right]$$

$$= \frac{\gamma_w \cdot G_a^* \cdot d}{\gamma_w}\left[22\% - 17\%\right]^*$$

d = depth of root zone = 90 cm

$$= 1.5 \times 90 \text{ cm } [5\%] = \frac{1.5 \times 90 \times 5}{100} = \mathbf{6.75\ cm}$$

Water depth reqd in the field

$$= \frac{6.75 \text{ cm}}{0.6} = 11.25 \text{ cm.} \quad \textbf{Ans.}$$

Depth of water available to plants for evapotranspiration = 6.75 cm

Consumptive use = 2.2 cm/day

2.2 cm of water depth is utilised by plants in = 1 day

$\therefore$ 6.75 cm of water depth will be utilised by plants in $= \frac{1}{2.2} \times 8.75$ day

= 3.07 days.

Hence, 11.25 cm water depth will be supplied to fields at every 3 days, which represents the irrigation frequency. **Ans.**

Q6. (*a*) *For a station 1, the recorded annual 24-hr maximum rainfall are given below. Compute the necessary data to estimate : (a) 24 hr maximum rainfall with a specified return period ; and (b) probability of a rainfall of not less than a specified depth occurring in 24 hours at station 1. Indicate the above features on a sketch (not necessarily plotted to exact data) prescribing the coordinate parameters.*

1 Year	1960	61	62	63	64	65	66	67	68	69
2 Rainfall, cm	14.2	13.1	7.5	14.9	12.4	14.2	7.0	16.1	8.3	9.2

1 Year	70	71	72	73	74	75	76	77	78	79	80
2 Rainfall cm	7.0	13.4	7.5	14.0	15.2	6.5	9.7	13.1	8.8	14.7	11.2

Solution. Please follow similar solved example 7.9 and compute Recurrence interval (T) values for all the given precipitations arranged in decreasing order, as shown in table below, using Eq. $T = \frac{N}{m}$, where N = Total no. of years of record = 21, and m is the ranking of storm :

* G_a is apparent sp gravity, which is the sp gravity of dry soil.

Solution Table for Q6 (a) of Year 2004

Year	*Maximum 24 hr rainfall in cm (arranged in decreasing order)*	*Ranking of storm (m)*	*Recurrence interval (T) in yrs.* $T = \frac{N}{m} = \frac{21}{col.\ (3)}$
(1)	*(2)*	*(3)*	*(4)*
1967	16.1	1	21.00
1974	15.2	2	10.5
1963	14.9	3	7.00
1979	14.7	4	5.25
1960	14.2	5	4.20
1965	14.2	6	3.50
1973	14.0	7	3.00
1971	13.4	8	2.63
1977	13.1	9	2.33
1961	13.1	10	2.10
1964	12.4	11	1.91
1980	11.2	12	1.75
1976	9.7	13	1.62
1969	9.2	14	1.50
1978	8.8	15	1.40
1968	8.3	16	1.31
1972	7.5	17	1.24
1962	7.5	18	1.17
1970	7.0	19	1.11
1966	7.0	20	1.05
1975	6.5	21	1.00

A graph can now be plotted between values of T (on log scale on x-axis) and values of rainfall (cm) on log scale on Y-axis using log-log graph paper. (Refer Fig. 7.30).

(b) To find probability of a rainfall of not less than a specified depth, say D cm, we can read out Recurrence interval (T) value for the given rainfall depth from the plotted graph. The inverse of this T value will give the required probability value p $\left(\because\ p = \frac{1}{T} \right)$.]

Q6. (b) *Draw a schematic diagram (in plan view) of a barrage on an alluvial river, with provision for overflow and fish passage, for diversion of flow into a main canal, including for sediment extraction. On the diagram, label the several features and the schematic flow paths.*

Q7. (a) *Ordinates of a 4-hr unit hydrograph (u.h.g.) on a catchment are tabulated. Derive the ordinates of a 2-hr u.h.g. for the same catchment.*

Time (hr)	0	4	8	12	16	20	24	28
Ordinate of 4-hr u.h.g. (m^3/s)	0	23	54	60	25	14	6	4

(20)

[**Hint.** Please follow similar solved example 7.52.]

Q8. (a) *In a subsurface pipe drainage system, it is desired to keep the highest level of the water table at 1.5 m below the ground surface. The depth of impervious layer from land surface is 10.0 m and the depth of the drain below the land surface is 2.0 m. The mean annual rainfall in the area is 96 cm ; and the coefficient of permeability is 6 $\times 10^{-6}$ m/s. Design the spacing the the drain pipes.* (20)

Solution. Please follow similar solved example 6.2 and compute as under :

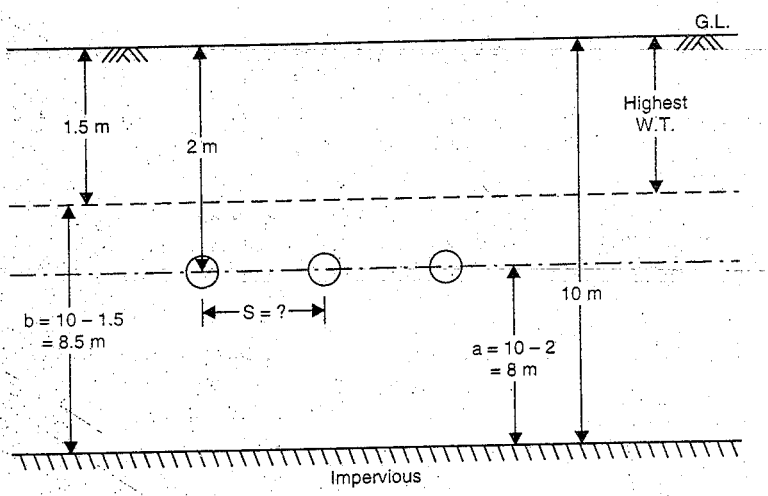


Fig. 31.14

From Fig. 31.14, we have

$a = 10 - 2 = 8$ m

$b = 10 - 1.5 = 8.5$ m

$K = 6 \times 10^{-6}$ m/s

P_{AA} = Annual rainfall = 96 cm = 0.96 m

$S = ?$

We have to use eqn. (6.9) as :

$$q = \frac{4K}{S}\left[b^2 - a^2\right]$$

Where q is the infiltration discharge into the ground, which should be removed by the drains in m^3/sec.

Assuming that 1% of the average annual rainfall of the considered place, shall be removed by the tile drains in 24 hours (a common assumption), we get

$$q = \frac{P_{AA} \times 1\%}{24 \times 60 \times 60} \times (S \times 1) \text{ cumec/m length of drains}$$

or
$$q = \frac{0.01 \times 0.96 \text{ m}}{24 \times 60 \times 60} \times \left[S \times 1\right] \text{ m}^3\text{/s per m length of drains} \quad ...(i)$$

Also
$$q = \frac{4 \times 6 \times 10^{-6}}{S}(8.5^2 - 8^2) \quad ...(ii)$$

Equating (*i*) and (*ii*), we get

$$\frac{4 \times 6 \times 10^{-6} \times 8.25}{S} = \frac{0.01 \times 0.96 \times S}{24 \times 3600}$$

or $$S^2 = \frac{4 \times 6 \times 10^{-6} \times 8.25 \times 24 \times 3600}{0.01 \times 0.96} = 1782 \text{ m}^2$$

or $$S = \mathbf{41.21 \text{ m.}} \quad \mathbf{Ans.}$$

Q. 8. (b) (i) In single as well as multiple purpose water resources projects, how are benefits classified ? Give brief explanation for each ; also indicate the distinction between "with/without" and "before/after" considerations.

Objective Questions of Engineering Services Exams.

Year 1993 (*Objective Paper II of Engg. Services*)*

Q. 1. The ordinates of a 3-hour hydrograph for a small catchment are given below:

Time (hour)	*Unit graph ordinates (m^3/s)*
0	0
3	1.5
6	4.5
9	18.6
12	12.0
15	9.4
18	4.6
21	2.3
24	0.8
27	0

If the design storm produces net rainfall depths of 4.6 cm and 3.5 cm in successive unit periods, and if the base flow is 2 m^3/s, then the peak flood flow (in m^3/s) will nearly be :

(*a*) 270 (*b*) 130

(*c*) 90 (*d*) 86.

Q. 2. A 4-hour rainfall in a catchment of 250 km^2 produces rainfall depths of 6.2 cm and 5.0 cm in successive 2-hour unit periods. Assuming the ϕ-index of the soil to be 1.2 cm/hour, the runoff volume in ham will be:

(*a*) 16 (*b*) 22

(*c*) 1600 (*d*) 2200.

* U.P.S.C. has, with effect from the year 1993, allowed students to take back home the Objective Papers, which it was not allowing earlier.

Q. 3. *A 3-hour unit hydrograph (UH) differs from a 6-hour U.H., for any catchment, in that :*

(*a*) both the time of rise and the peak ordinate for the former are less than that for the latter

(*b*) both the time of rise and the peak ordinate for the former are greater that that of the latter

(*c*) the time of rise is less but the peak ordinate is greater for the former as compared to that of the latter

(*d*) the time of rise is greater but the peak ordinate is less for the former as compared to that of the latter.

Q. 4. *As per the recommendations of the ISB (NBS), the shape of a lined canal is:*

(*a*) trapezoidal (*b*) semicircular
(*c*) parabolic (*d*) elliptic.

Q. 5. *Which of the following pairs are correctly matched ?*

1.	Device to receive and eject drainage from canal	inlet and outlet
2.	Cross-drainage structure when canal bed level and drain bed level are same	canal siphon
3.	Cross-masonary work to facilitate road transport.	bridge

Codes :

(*a*) 1, 2 and 3 (*b*) 1 and 3
(*c*) 2 and 3 (*d*) 1 and 2.

Q. 6. Application efficiency of irrigation is best defined as the ratio of:

(*a*) water for normal consumptive requirement to water depleted from the root zone

(*b*) water actually stored in the root zone to water delivered to the farm

(*c*) water reaching the farm to water released from the head works

(*d*) water actually delivered to the farm to water actually reaching the farm.

Q. 7*. Match List I with List II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Contour bunding	1. High rainfall
B. Bench terrace with outward slope	2. Low rainfall
C. Land slope = 100%	3. 45°
D. Drop inlet spillway	4. 90°
	5. Gully control.

Codes :

	A	*B*	*C*	*D*
(*a*)	1	2	3	4
(*b*)	3	5	2	1
(*c*)	2	1	3	5
(*d*)	3	4	5	1.

ANSWERS

1. (*b*) **2.** (*c*) **3.** (*c*) **4.** (*a*)
5. (*b*) **6.** (*b*) **7.** (*c*).

Year 1994 *(Objective Paper II of Engineering Services)*

Q. 1. Consider the following statements regarding confined aquifers :

1. The aquifer is bound at top and below by impervious strata
2. The pressure of water is greater than atmospheric pressure
3. A tubewell sunk in such an aquifer starts flowing always by itself.
4. The aquifer is fully saturated .

Of the above statements :

(*a*) 1, 2 and 3 are correct (*b*) 2, 3 and 4 are correct
(*c*) 1, 2 and 4 are correct (*d*) 1, 3 and 4 are correct.

Q. 2. The given diagram I shows the curve of a hydrograph. Which of the following would cause the peak of the curve to shift to the right ?

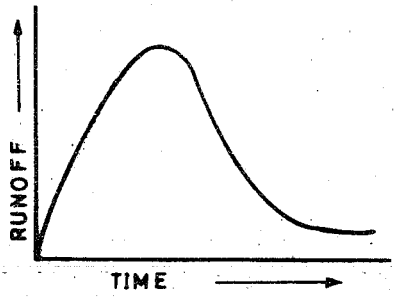


Diagram I. (Fig. 32.1)

(1) When the length of the overland flow is more
(2) When the slope of the land surface is less
(3) When the runoff is more
(4) When the rainfall is moderate.

Select the correct answer using the codes given below :

Codes :

(*a*) 3 and 4 (*b*) 2 and 4
(*c*) 1 and 2 (*d*) 2, 3 and 4.

Q. 3. A tropical cyclone in the northern hemisphere is a zone of :

(*a*) low pressure with clockwise wind
(*b*) low pressure with anticlockwise wind
(*c*) high pressure with clockwise wind
(*d*) high pressure with anticlockwise wind.

Q. 4. If one wants to be 90% sure that the design flood in a dam project will not occur during the design life period of 100 years, the recurrence interval for such a flood would be :

(*a*) about 90 years (*b*) equal to 100 years

(*c*) about 110 years (*d*) roughly 1000 years.

Q. 5.* For the estimation of a flood with a return period of T years by Gumbel's method, which of the following sets of data regarding the annual flood series would be required ?

(*a*) Mean and Standard deviation (SD) of the flood flow series

(*b*) Mean flood flow, SD of the flood flow series, and the Length of record

(*c*) SD of the flood flow series and Length of record

(*d*) Mean, SD, and coefficient of skew of the flood flow series.

Q. 6.* A channel designed by Lacey's theory has a mean velocity of one m/s. The silt factor is unity. The hydraulic mean radius will be :

(*a*) 2.5 m (*b*) 2 m

(*c*) 1 m (*d*) 0.5 m.

Q. 7. If D = scour depth measured below high flood level, and d = depth of scour below the original bed level, then the width of the launching apron at toe of guide bank will generally be :

(*a*) 1.5*D* (*b*) 1.5*d*

(*c*) 2.0 *D* (*d*) 2.0 *d*.

Q. 8. Match list I with list II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Land capability	1. Drip irrigation
B. Sandy soil	2. Parshall flume
C. Consumptive use of a crop	3. Contour strip cropping
D. Water flow measurement	4. Blanly Criddle formula
	5. U.S. Bureau of reclamation classification.

Codes :

	A	*B*	*C*	*D*
(*a*)	5	1	4	2
(*b*)	1	2	4	3
(*c*)	5	3	1	2
(*d*)	1	2	5	3

ANSWERS

1. (*c*)	**2.** (*c*)	**3.** (*b*)	**4.** (*d*)
5. (*d*)	**6.** (*a*)	**7.** (*b*)	**8.** (*a*)

Hints for Solving Starred Questions of Year 1994

Q. 5. Gumbel's method uses the eqn :

$$Q_p = \overline{Q}_p + k\,\sigma$$

where $\overline{Q}_p$ = Mean flood flow

k = Coefficient of skew

σ = Standard Deviation.

Hence for estimation of flood, we need Mean flood flows, S.D. (Standard Deviation) and Coefficient of skew (k). Hence, choice (d) is correct.

Q. 6. Use eqn. 4.15 as :

$$R = \frac{5}{2}\left(\frac{v^2}{f}\right) = \frac{5}{2}\left(\frac{1^2}{1}\right)$$

$$= 2.5 \text{ m}.$$

Hence choice (a) is correct. **Ans.**

Year 1995 *(Objective Paper II of Engineering Services)*

Q. 1. Which one of the following constitutes the basic assumption of the unit hydrograph theory :

(a) Non-linear response and time invariable.

(b) Non-linear time variance and linear response

(c) Linear response and linear time variance

(d) Time invariance and linear response.

Q. 2. The following parameters relate to the design of weirs of permeable foundations

1. Scour depth
2. Exit gradient
3. Uplift pressure
4. Unbalanced head.

Design of the downstream end pile of the weir depends upon :

(a) 1 and 2 (b) 1 and 4

(c) 2 and 3 (d) 3 and 4.

Q. 3. Given that the base period is 100 days and the duty of the canal is 1000 hectares per cumec, the depth of water will be

(a) 0.864 cm (b) 8.64 cm

(c) 86.4 cm (d) 864 cm.

Q. 4. The most economical method of soil conservation is to :

(a) construct check dams (b) construct contour bunds

(c) drain the soil (d) aforest the soil.

Q. 5. A pumped storage plant is a

(a) high head plant (b) runoff river plant

(c) peak load plant (d) base load plant.

ANSWERS

1. (*d*) **2.** (*a*) **3.** (*c*) **4.** (*d*) **5.** (*c*).

Year 1996 (*Objective Paper II of Engineering Services*)

Q. 1. The percentage standard error of precipitation averages is often expressed functionally or graphically in terms of (*i*) precipitation gauge network density expressed as area per gauge ; and (*ii*) total area of catchment. The percentage standard error

(*a*) increases with area per gauge as well as with total area

(*b*) decreases with area per gauge as well as with total area.

(*c*) increases with area per gauge but decreases with total area.

(*d*) decreases with area per gauge but increases with total area.

Q. 2. If '*p*' the precipitation, '*a*' is the area represented by a raingauge, and *n* is the number of rainguages in the catchment area, then the weighted mean rainfall is :

(*a*) $\frac{\Sigma ap^3}{\Sigma a^2}$ (*b*) $\frac{\Sigma ap}{n}$

(*c*) $\frac{\Sigma ap}{\Sigma a}$ (*d*) $\frac{\Sigma ap^5}{\Sigma a^3}$

Q. 3. Depth-Area-Duration curves of precipitation are drawn as

(*a*) Minimising envelopes through the appropriate data points.

(*b*) Maximising envelopes through the appropriate data points.

(*c*) best fit mean curves through the appropriate data points.

(*d*) best fit mean straight lines through the appropriate data points.

Q. 4. Match the list I with list II and select the correct answer using the codes given below the lists :

List I	*List II*
A. Conservation reservoirs	1. Uncontrolled outlets
B. Retarding basins	2. Flood-fighting
C. Flood plains	3. Temporary storage of flood water
D. Flood walls	4. Controlled outlets

Codes :

(*a*)	A	B	C	D
	1	4	3	2
(*b*)	A	B	C	D
	1	4	2	3
(*c*)	A	B	C	D
	4	1	3	2
(*d*)	A	B	C	D
	4	1	2	3

Q. 5. The following four hydrological features have to be estimated or taken as inputs before one can compute the flood hydrograph at any catchment outlet.

1. Unit hydrograph 2. Rainfall hydrograph
3. Iufiltration index 4. Base flow.

The correct order in which they have to be employed in the computations is

(*a*) 1, 2, 3, 4 (*b*) 2, 1, 4, 3
(*c*) 2, 3, 1, 4 (*c*) 4, 3, 2, 1

Q.6. In the alignment of an irrigation channel, wherefrom off-takes have to be provided at regular intervals, changes in the given channel parameters are made use of. The correct sequence of the decreasing order of preference of these parameters is

(*a*) width, slope, depth (*b*) width, depth, slope
(*c*) depth, slope, width (*d*) depth, width, slope.

Q.7. Match List I will List II and select the correct answer using the codes given below the lists

	List I *(Indications of terms)*	*List II* *(Terms)*
A.	Tail water curve much above jump height curve	1. Basin at bed level
B.	Tailwater curve slightly above jump height curve	2. Sunk basin
C.	Tailwater curve coinciding with jump height curve	3. roller bucket
D.	Tailwater curve below the jump height curve	4. Sloping apron

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	3	4	1	2
(*c*)	4	3	1	2
(*d*)	3	4	2	1

Q. 8. The yield of a well depends upon

(*a*) permeability of soil
(*b*) area of aquifer opening into the wells
(*c*) actual flow velocity
(*d*) all of the above

Q. 9. The construction of impounding reservoir is required when

(*a*) average annual flow in the stream is lower than average demand
(*b*) the rate of flow in the stream in dry season is more than the demand
(*c*) the rate of flow in the stream in dry season is less than the demand
(*d*) the rate of flow in the stream is equal to the demand.

ANSWERS

1. (*a*) **2.** (*c*) **3.** (*c*) **4.** (*c*) **5.** (*c*)
6. (*b*) **7.** (*c*) **8.** (*a*) **9.** (*c*)

Year 1997 (*Objective Paper II of Engineering Services*)

Q. 1*. A channel of bed slope 0.0009 carries a discharge of 30 m^3/s when the depth of flow is 1.0 m. What is the discharge carried by an exactly similar channel at the same depth of flow if the slope is increased to 0.0001 ?

(*a*) 10 m^3/s (*b*) 15 m^3/s
(*c*) 60 m^3/s (*d*) 90 m^3/s

Q. 2*. At a hydraulic jump, the depths at the two sides are 0.4 m and 1.4 m. The head loss in the jump is nearly

(*a*) 1.0 m (*b*) 0.9 m
(*c*) 0.7 m (*d*) 0.45 m

Q. 3. Which one of the following pairs relating to flumes carrying open channel flow is correctly matched ?

(*a*) Non-modular flume ... Flow is unaffected by drowning
(*b*) Venturi flume ... Standing wave forms at the throat.
(*c*) Venturi flume ... Flow at the throat is less than critical velocity.
(*d*) Standing wave flume ... Hump is not provided at the throat.

Q. 4*. Mean precipitation over an area is best obtained from gauged amounts by

(*a*) arithmetic mean method
(*b*) Thiessen method
(*c*) linearly interpolated isohyetal method
(*d*) Orographically weighed isohyetal method.

Q. 5. The following steps are involved in arriving at a unit hydrograph.

1. Estimating the surface runoff in depth
2. Estimating the surface runoff in volume
3. Separation of base flow
4. Dividing surface runoff ordinates by depth of runoff

The correct sequence of these steps is

(*a*) 3, 2, 1, 4 (*b*) 2, 3, 4, 1
(*c*) 3, 1, 2, 4 (*d*) 4, 3, 2, 1

Q. 6*. Probability of a 10 year flood to occur at least once in the next 4 years is

(*a*) 25% (*b*) 35%
(*c*) 50% (*d*) 65%

Q. 7. The standard Project flood is

(*a*) derived from the probable maximum precipitation in the region
(*b*) derived from the severemost meteorological conditions any where in the country.
(*c*) the flood with return period of 100 years.
(*d*) the same as the probable maximum flood.

Q. 8. Match List I (control structures) with List II (Functions of the control structures) and select the correct answer using the codes given below the Lists.

List I	*List II*
A. Canal drop	1. Control of flow depth
B. Canal Escape	2. Control of bed grade
C. Canal cross-regulator	3. Control of full supply level
D. Canal outlets.	4. Control of discharge

Codes :

	A	B	C	D
(*a*)	2	3	4	1
(*b*)	2	3	1	4
(*c*)	3	2	1	4
(*d*)	3	2	4	1

Q. 9*. For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be

(*a*) 0.30 (*b*) 0.45

(*c*) 0.70 (*d*) 1.32

Q. 10. Match List I (Main provision) with List II (surplussing arrangement) and select the correct answer using the codes given below the Lists.

List I	*List II*
A. Minor Irrigation work	1. Saddle spillway
B. Medium irrigation project in interior area	2. Syphon spillway
C. Earth dam across main river	3. Ogee spillway
D. Masonry dam on good rock	4. Surplus weir.

Codes

	A	B	C	D
(*a*)	4	2	1	3
(*b*)	4	2	3	1
(*c*)	2	4	3	1
(*d*)	2	4	1	3

Q. 11. While considering weir designs on permeable soils, the correction for mutual interference of sheet piles is NOT applicable on an intermediate pile if the outer pile :

(*a*) goes deeper than the intermediate pile and is farther from the intermediate pile by more than twice its own length

(*b*) goes only just as deep as the intermediate pile and is within a distance of one and half times its own length

(*c*) does not go as deep as the intermediate pile, no matter what the horizontal distance between them is

(*d*) is safe against deleterious exit gradient

Q. 12*. The following data were recorded from an irrigation field ;

1. Field capacity = 20%
2. Permanent wilting point = 10%
3. Permissible depletion of available soil moisture = 50%
4. Dry unit weight of soil = 1500 kgf/m^3
5. Effective rainfall = 25 mm

Based on these data, the net irrigation requirement per metre depth of soil will be

(*a*) 75 mm (*b*) 125 mm
(*c*) 50 mm (*d*) 25 mm

Q. 13. The correct sequence, in the direction of the flow of water for installations in a hydropower plant is

(*a*) reservoir, surgetank, turbine, penstock
(*b*) reservoir, penstock, surge tank, turbine
(*c*) reservoir, penstock, turbine, surge tank
(*d*) reservoir, surge tank, penstock, turbine

ANSWERS

1. (*a*)	**2.** (*d*)	**3.** (*b*)	**4.** (*d*)	**5.** (*a*)	**6.** (*b*)
7. (*b*)	**8.** (*b*)	**9.** (*c*)	**10.** (*a*)	**11.** (*b*)	**12.** (*c*)
13. (*b*)					

Hints for Solving Starred Questions of Year 1997

Q. 1. $Q = \frac{1}{n} \cdot A.R^{2/3} \cdot \sqrt{S}$

$$\therefore \quad Q_2 = \sqrt{\frac{0.0001}{0.0009}} \cdot Q_1 = \frac{1}{3} \times 30\ \text{m}^3/\text{s} = 10\ \text{m}^3/\text{s}\ \text{Ans.}$$

Q. 2. $H_L = \frac{(y_2 - y_1)^3}{4 \cdot y_1 y_2} = \frac{(1.4 - 0.4)^3}{4 \times 0.4 \times 1.4} = 0.45\ \text{m}$ Ans.

Q. 4. The Isohyetal method for determining mean rainfall in a catchment is superior to the arithmetic mean method as well as 'Thiessans' Polygon method, especially when the stations are large in number and the catchment is large (more than 5000 sqkm or so). The isohyetal method also permits consideration of topographic effects, and is therefore more accurate in mountainous areas. In this method, the average value of rainfall indicated by two isohyets is assumed to be acting over the inter isohyet area. The isohyetal method thus considers different rainfalls for different part areas, which amounts to orographical weighing of the part areas.

Q. 5. When the river runoff graph is known, the base flow has to be first of all subtracted from it to obtain S.R.O. graph. The volume of S.R.O. is then computed from the S.R.O. graph. The S.R.O. volume is then divided by catchment area to obtain S.R.O. depth. The S.R.O. ordinates are finally

divided by S.R.O. depth (in cm) to obtain ordinates of unit hydrograph. Hence, the sequence 3, 2, 1, 4 is correct. **Ans.**

Q. 6. $T = 10$ yrs

$$p = \frac{1}{T} = \frac{1}{10} = 0.1$$

$$R = \left[1 - (1 - 0.1)^4\right]$$

$$= 1 - (0.9)^4 = 0.3439 \approx 0.35 \text{ or } 35\% \quad \textbf{Ans.}$$

Q. 9. $f = 1.75 \cdot \sqrt{dmm}$

$$= 1.75\sqrt{0.16} = 1.75 \times 0.4 = 0.70 \quad \textbf{Ans.}$$

Q. 12. Available m.c. = Field capacity m.c. – wilting Pt. m.c.

$$= 20\% - 10\% = 10\%$$

Readily available m.c. = 50% × Available m.c.

$$= 50\% \times 10\% = 5\%$$

Optimum m.c. = 20% – 5% = 15%

Irrigation water is to be supplied when m.c. falls to 15% and is to be recouped upto 20%

$$= \frac{\gamma_d}{w}\left[\text{Field m.c.} - \text{Optimum m.c.}\right]$$

For 1 m depth of soil, the water reqd. to recoup m.c. from 15% to 20%

$$= \frac{1500}{1000} \times 1 \text{ m } [20\% - 15\%]$$

$$= 1.5 \times 5\% \text{ m} = 1.5 \times 0.05 = 0.075 \text{ m} = 75 \text{ mm}$$

Irrigation water reqd. = Total water reqd. – Effective Rainfall

$$= 75 \text{ mm} - 25 \text{ mm} = 50 \text{ mm} \quad \textbf{Ans.}$$

Year 1998 (*Engineering Services*)

Q. 1. Depth-area-duration curves would seem to resemble

(*a*) arcs of a circle concave upwards with duration increasing outward

(*b*) first quadrant limbs of hyperbolae with duration increasing outward

(*c*) third quadrant limbs of hyperbolae with duration decreasing outward

(*d*) first quadrant limbs of hyperbolae with duration decreasing outward

Q. 2. When a river starts meandering, the sediment carrying capacity

(*a*) first decreases and ultimately increases

(*b*) first increases and ultimately decreases

(*c*) remains unaffected as the plan changes continue

(*d*) changes erratically all the time leaving permanent braids

Q. 3. Asubmerged pipe outlet from a minor feeds into a well chamber across the bank. Water from the well chamber overflows a weir- like face into the field channel. The flow into the field channel

(*a*) will increase with any drop in the water level in the minor

(*b*) will not be much affected by any change in the water level in the minor
(*c*) will decrease with any drop in the water level in the minor
(*d*) will be proportional to the flow in the minor when the minor flows between set ranges

Q. 4. Match List-I (Types of turbines) with List-II (Ranges of specific speeds in MKS units) and select the correct answer using the codes given below the lists :

List I	*List II*
A. Francis	1. 10 – 35
B. Kaplan	2. 35 – 60
C. Pelton with one jet	3. 60 – 300
D. Pelton with two jets	4. 300 – 1000

Codes :

	A	B	C	D
(*a*)	3	4	2	1
(*b*)	4	3	2	1
(*c*)	3	4	1	2
(*d*)	4	3	1	2

ANSWERS

1. (*a*) **2.** (*c*) **3.** (*c*) **4.** (*c*)

Year 1999 (*Engineering Services*)

Q. 1.* To generate 10,000 hp under a head of 81 m while working at a speed of 500 rpm, the turbine of choice would be :

(*a*) Pelton (*b*) Kaplan
(*c*) Bulb (*d*) Francis.

Q. 2. Consider the following statements :

1. Run of river plants can be located on any river
2. Runaway speed of a turbine is generally 180% of normal speed.
3. Underground power stations are suited to areas susceptible to land slides.
4. Higher the specific speed, higher will be the discharge and head.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 2 and 3
(*c*) 1 and 4 (*d*) 2 and 4

Q. 3. Match List I and II and select the correct answer using the codes given below the lists

List I (Turbine)		*List II (Specific speed)*	
A	Pelton	1	25
B	Propeller	2	75
C	Kaplan	3	500
D	Francis	4	800
		5	900

Codes

	A	*B*	*C*	*D*
(*a*)	1	4	5	2
(*b*)	3	4	2	5
(*c*)	4	1	5	3
(*d*)	1	5	4	2

Q. 4.* An aquifer confined at top and bottom by impermeable layers is stratified into three layers as follows.

Layer	*Thickness (m)*	*Permeabilaty m/day*
Top layer	4	30
Middle layer	2	10
Bottom layer	6	20

The transmissivity (m^2/day) of the aquifer is

(*a*) 260 (*b*) 227
(*c*) 80 (*d*) 23

Q. 5. The spacing of tile drains to relieve water-logged land is directly proportional to the

(*a*) depth of drain below the ground surface
(*b*) depth of impervious strata from the drain
(*c*) depth of drain below the water level.
(*d*) coefficient of permeability of the soil to be drained.

Q. 6.* The maximum permissible suction lift for centrifugal pump *in practice* (at sea level and at 30°C) is :

(*a*) 12 m (*b*) 10 m
(*c*) 6 m (*d*) 3 m.

Q. 7.* Consider the following statements :

1. Pumps in series operation allow the head to increase.
2. Pumps in series operation increase the flow rate.
3. Pumps in parallel operation increase the flow rate.
4. Pumps in parallel operation allow the head to increase.

Which of these statements are correct ?

(*a*) 1 and 3 (*b*) 1 and 4
(*c*) 2 and 4 (*d*) 3 and 4.

Q. 8. In a flow-mass curve study, the demand line drawn from a ridge does not intersect the mass curve again. This implies that

(*a*) the reservoir is not full at the beginning

(*b*) the storage is not adequate.

(*c*) the demand cannot be met by the inflow, as the reservoir will not refill

(*d*) the reservoir is wasting water by spill.

Q. 9.* The following rainfall data refers to stations 'A' and 'B' which are equidistant from station 'X'.

	Station 'A'	*Station 'X'*	*Station 'B'*
Long term normal annual rainfall in mm	200	250	300
Annual rainfall in mm for the year 1940	140	P	270

The value of P will be :

(*a*) 250 (*b*) 220

(*c*) 205 (*d*) 200

Q. 10. Consider the following statements :

1. A 100-year flood discharge is greater than a 50 year flood discharge.

2. 90% dependable flow is greater than 50% dependable flow

3. Evaporation from salt-water surface is less than that from fresh water surface.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 2 and 3

(*c*) 1 and 3 (*d*) 1, 2 and 3

Q. 11. The trap efficiency of a reservoir is a function of :

(*a*) inflow in the reservoir (*b*) ratio of inflow to storage capacity

(*c*) ratio of reservoir capacity to inflow. (*d*) reservoir capacity.

Q. 12. If a 4-hour unit hydrograph of a certain basin has a peak ordinate of 80 m^3/s, the peak ordinate of a 2-hour unit hydrograph for the same basin will be :

(*a*) equal to 80 m^3/s (*b*) greater then 80 m^3/s

(*c*) less than 80 m^3/s (*d*) between 40 m^3/s to 80 m^3/s

Q. 13. Match List I (Name of scientist) with List II (Contribution to field of hydrology) and select the correct answer using the Codes given below the lists :

List I	*List II*
A. Dalton	1. Unit hydrograph
B. Snyder	2. Evaporation
C. Blaney Criddle	3. Empirical flood formula
D. Sherman	4. Synthetic unit hydrograph
	5. Consumptive use equation

Codes

	A	*B*	*C*	*D*
(*a*)	2	3	5	1
(*b*)	1	4	3	2
(*c*)	2	4	5	1
(*d*)	1	3	5	4

Q. 14.* In a linear reservoir, the

(*a*) volume varies linearly with elevation

(*b*) outflow rate varies linearly with storage

(*c*) storage varies linearly with time

(*d*) storage varies linearly with inflow rate

Q. 15. A culvert is designed for a peak flow Q_p on the basis of rational formula. If a storm of the same intensity as used in the design and twice the duration occurs, then the resulting peak discharge will be :

(*a*) Q_p (*b*) $\frac{Q_p}{2}$

(*c*) $\sqrt{2}\,Q_p$ (*d*) $2\,Q_p$.

Q. 16. Match List I with List II and select the correct answer using the codes given below the list

List I	*List II*
A. Specific yield	1. Volume of water retained per unit volume of aquifer
B. Specific capacity	2. Volume of water drained by gravity per unit volume of aquifer
C. Specific retention	3. Difference of porosity and specific storage.
D. Specific storage	4. Well yield per unit drawdown
	5. Volume of water released from unit volume of aquifer for unit decline in piezometric head.

Codes

	A	*B*	*C*	*D*
(*a*)	2	4	1	5
(*b*)	4	2	3	5
(*c*)	2	5	1	4
(*d*)	4	2	3	1

Q. 17. The best design of an arch dam is when :

(*a*) All horizontal water loads are transferred horizontally to the abutments.

(*b*) The dam is safe against sliding at various levels

(*c*) The load is divided between the arches and cantilevers, and the deflections at the conjugal points being equal.

(*d*) The deflections of the cantilevers are equal at different points.

ANSWERS

1. (*d*)	**2.** (*b*)	**3.** (*a*)	**4.** (*b*)	**5.** (*d*)	**6.** (*c*)
7. (*a*)	**8.** (*c*)	**9.** (*d*)	**10.** (*c*)	**11**(*c*).	**12.** (*b*)
13. (*c*)	**14.** (*b*)	**15.** (*a*)	**16.** (*a*)	**17.** (*c*)	

Hints for Solving Starred Questions of the Year 1999

Q. 1. From Eq. (24.1), we have

$$N_s = \frac{N.\sqrt{P_t}}{H^{5/4}}$$

where N = Normal speed of turbine in RPM = 500

P_t = Turbine output in metric horse power = 10,000 hp.

H = Effective head in m = 81 m

$$\therefore \quad N_s = \frac{500\sqrt{10,000}}{(81)^{5/4}} = 205.76$$

From Table 24.1, the turbine which is suitable for a head between 25–450 m and N_s of 60 – 300 is Francis turbine; and hence the recommended turbine in this case is Francis type (Reaction type). Hence, choice (*d*) is correct. **Ans.**

Q. 4. For non-homogeneous layered aquifers, we have to determine effective permeability coefficient by using equation (5.20) given in "Soil Mechanics and Foundation Engineering" by the same author, for vertical flow, as :

$$K_v = \frac{d_1 + d_2 + d_3}{\frac{d_1}{K_1} + \frac{d_2}{K_2} + \frac{d_3}{K_3}}$$

$$= \frac{4+2+6}{\frac{4}{30} + \frac{2}{10} + \frac{6}{20}} = 12 \times \frac{60}{38} \text{ m/day}$$

$$T = K.d = 12 \times \frac{60}{38} \times (4+2+6)$$

$$= 12 \times \frac{60}{38} \times 12 \text{ m}^2/\text{day} = 227 \text{ m}^2/\text{day}.$$

Hence, choice (*b*) is correct **Ans.**

Q. 6. The max. permissible suction lift for a centrifugal pump is given by the equation :

$$H_s = H_a - H_f - e_s - NPSH - F_s \qquad ...(32.1)$$

where, H_s = Max. Practical suction lift, *m*

H_a = Atmospheric pressure at the water surface = 10.33 m at sea level, with reduction of 0.36 m for each 300 m rise in altitude above sea level.

e_a = saturated vapour pressure of water = 4.609 kN/m^2 at 30° C(from Standard Physical tables — Appendix Table A–1)

$NPSH$ = Net positive suction head of the pump, including losses in the impeller& vel. head. It is calculated from the characteristic curves of the given pump, supplied by the manufacturer.

F_s = Factor of safety, which is usually taken as 0.6 m.

From the above eqn., we can easily infer that $H_s < H_a$, while $H_a < 10$ m. For this reason, for all practical proposes, the suction lift for centrifugal pumps is usually limited to 6 – 8 m, as stated in our text. In the given question, the choice below 10m is given to be 6m. Hence, we mark this choice at (*c*) as the correct choice. **Ans.**

Q. 7. When two or more pumps are connected in series, such as in multi-stage pumping operations, the discharge from the first stage is pumped to the second stage, and so on. The same discharge passes through all the stages. This arrangement becomes essential when the reqd. head is more than that can be produced with a single pump. Each stage in such pumping operations, adds an additional head to the flow. Thus, the combined head of two pumps operating in series is equal to the sum of the individual heads, for a specific discharge.

Sometimes, water pumps are installed in parallel as in the case of pumping water from a sump well using more than one pump. The head remains constant. The combined discharge of the two pumps operating in parallel is equal to the sum of the individual discharges, for a specific head.

From the above discussion, it can be very clearly inferred that the statement 1 and 3 are only true, and hence choice (*a*) is correct. **Ans.**

Q. 9. $N_x = 250$

$$10\%\, N_x = 25$$

$$N_x + 10\%\, N_X = 275 < 300\ (N_B)$$

Hence, use weighted mean method to compute the missing rainfall data P_X.

$$\therefore \quad P_x = \frac{1}{2}\left[\frac{N_X}{N_A}\cdot P_A + \frac{N_X}{N_B}\cdot P_B\right]$$

$$= \frac{1}{2}\left[\frac{250}{200}\times 140 + \frac{250}{300}\times 270\right] = 200$$

Hence, choice (*d*) is correct. **Ans.**

Q. 14. Please see chapter on 'Flood Forecasting' in "*Hydrology and Water Resources Engg.*" by the same author, Page 418, 11th edition, to tick the correct choice, which is given at (*b*) **Ans.**

Year 2000 (*Engineering Services*)

Q. 1. If the base period of a 6 hour unit hydrograph of a basin is 84 hours, then a 12 hour unit hydrograph derived from this 6 hour unit hydrograph will have a base period of

(*a*) 72 hours (*b*) 78 hours

(*c*) 84 hours (*d*) 90 hours.

Q. 2*. The stage in a river is 4.8 m; the water surface slope is 1 in 10,000 and the discharge in the stream is 600 m^3/s. If the stage remains the same and the water surface slope is 1 in 14,400, then the discharge in the stream will be :

(*a*) 300 m^3/s (*b*) 400 m^3/s

(*c*) 600 m^3/s (*d*) 500 m^3/s

Q. 3. If a tangent drawn parallel to the demand line from a ridge point of a mass curve dose not intersect the mass curve again, it can be inferred that the :

(*a*)frequency of the flood entering into the reservoir is less.
(*b*) inflow into the reservoir cannot meet the demand.
(*c*) reservoir is overflowing resulting in wastage.
(*d*) reservoir can meet higher demand.

Q. 4*. In order to ensure that no scouring takes place in the bed of a channel of bed slope '*S*' constructed on alluvial soil of particle size '*d*' cm, the flow velocity should be restricted to :

(*a*) $4.85\, d^{\frac{1}{2}} . S^{-\frac{1}{6}}$ (*b*) $4.85\, d^{-\frac{1}{2}} . S^{\frac{1}{6}}$
(*c*) $0.48\, d^{\frac{1}{2}} . S^{\frac{1}{6}}$ (*d*) $0.48\, d^{\frac{1}{2}} . S^{-\frac{1}{6}}$

Q. 5. Which one of the following sets is used to control the seepage through the foundations of an earth dam ?

(*a*) Chimney drain, upstream blanket and cut off trench
(*b*) Cut off sheet piles, upstream blanket and cut off trench.
(*c*) Upstream blanket, cut off sheet piles and chimney drains
(*d*) Relief wells, upstream blanket and chimney drain.

Q. 6. Consider the following statements :

1. In designing the hydraulic structure in permeable foundation by Khosla's theory, the slope correction is applicable to piles located at the ends of the sloped floor only in a structure with one or more sloped floors.
2. Khosla's theory can be correctly applied in alluvium of finite depth.
3. Length of flow has lesser effect on exit gradient than the depth of piles.
4. Intercepts between hydraulic gradient line and free water surface on the glacis and horizontal floor downstream is the unbalanced uplift in a structure.

Which of these statements related to the design of hydraulic structure are correct ?

(*a*) 1, 2 and 3 (*b*) 1, 3 and 4
(*c*) 2, 3 and 4 (*d*) 1, 2 and 4

Q. 7. Match *List-I* (Energy dissipation) with *List-II* (Water level and slope condition) and select the correct answer using the codes given below the lists :

List I	*List II*
A. Roller bucket	1. TWL (tail channel water level) is slightly above JWL (Jump height water level) and the slope of the channel is mild.
B. Ski-jump bucket	2. TWL is considerably above JWL and the slope of the channel is mild.
C. Standing wave basin with depressed floor	3. TWL is slightly below JWL and the slope of the channel is mild
D. Standing wave basin with raised floor	4. TWL is considerably less than JWL and the slope of the channel is steep.

Codes

	A	*B*	*C*	*D*
(*a*)	4	2	1	3
(*b*)	2	4	1	3
(*c*)	2	4	3	1
(*d*)	4	2	3	1

Q. 8. If the depth is 8.64 cm on a field over a base period of 10 days, then the duty is :

(*a*) 10 hectares per cum/s (*b*) 100 hectares per cum/s
(*c*) 864 hectares per cum/s (*d*) 1000 hectares per cum/s

Q. 9. Specific capacity of a well is the :

(*a*) volume of water that can be extracted by the force of gravity from a unit volume of aquifer.
(*b*) discharge per unit drawdown of the well.
(*c*) drawdown per unit discharge of the well.
(*d*) rate of flow through a unit width and entire thickness of aquifer.

ANSWERS

1. (*d*) **2.** (*d*) **3.** (*b*) **4.** (*a*) **5.** (*b*)
6. (*b*) **7.** (*c*) **8.** (*d*) **9.** (*b*)

Hints for Solving Starred Questions of the year 2000

Q. 2. $Q_1 = 600 = K \cdot \sqrt{S_{f_1}} = \dfrac{K}{\sqrt{10000}}$; $\quad Q_2 = ? = K \cdot \sqrt{S_{f_2}} = \dfrac{k}{\sqrt{14400}}$

$$\frac{Q_1}{Q_2} = \frac{600}{Q_2} = \frac{\sqrt{14400}}{\sqrt{10000}} = \frac{1}{1.2}; \; Q_2 = 500 \text{ m}^3/\text{sec}$$

Hence, choice (*d*) is correct. **Ans.**

Q. 4. For designing a non scouring alluvial channel, we know from eqn. (4.13) that

$$d \le 11 \text{ RS}$$

or $$R \le \frac{d}{11 S} \quad \text{or} \quad R_{max} = \frac{d}{11 S}$$

Also, $$V = \frac{1}{n} R^{2/3} \cdot \sqrt{S}$$

or $$V_{max} = \frac{1}{n} \cdot R_{max}^{2/3} \cdot \sqrt{S}$$

But $$= n = \frac{1}{24} \cdot d^{1/6} \text{ (Strickler's formula)}$$

$$\therefore \quad V_{max} = \frac{24}{d^{1/6}} \cdot \left(\frac{d}{11 S}\right)^{2/3} \cdot \sqrt{S} = \frac{24}{(11)^{2/3}} \cdot d^{2/3 - 1/6} \cdot S^{1/2 - 2/3}$$

or $$\boxed{V_{max} = 4.85 \cdot d^{1/2} \cdot S^{-\frac{1}{6}}}$$

Hence, velocity in a non scouring channel should be restricted to above eqn., which makes choice (*a*) as the correct choice. **Ans.**

Year 2001 (*Engineering Services*)

Q. 1. Match *List I* (Surface) with *List II* (Approximate range of Manning's n) and select the correct answer using the codes given below the lists :

List I	*List 2*
A. Gravel river bed with 50 mm diameter bed	1. 0.02 to 0.022
B. Badly maintained unlined irrigation channel with weed growth	2. 0.025 to 0.04
C. Concrete lined channel	3. 0.013 to 0.017
D. Channel with brick lining	4. 0.04 to 0.08

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	4	2	3	1
(*c*)	1	3	2	4
(*d*)	1	2	3	4

Q. 2. Which of the following principles relate to a unit hydrograph ?

1. The hydrographs of direct run off due to effective rainfall of equal duration have the same time base.
2. Effective rainfall is not uniformly distributed within its duration.
3. Effective rainfall is uniformly distributed throughout the whole area of drainage basin.
4. Hydrograph of direct run off from a basin due to a given period of effective rainfall reflects the combination of all the physical characteristics of the basin.

Select the correct answer using the codes given below :

Codes :

(*a*) 1, 2 and 3 (*b*) 1, 2 and 4

(*c*) 2, 3 and 4 (*d*) 1, 3 and 4.

Q. 3. Match *List I* with *List II* and select the correct answer using the codes given below the lists :

List I	*List II*
A. Anemometer	1. Humidity
B. Rain simulator	2. Evapotranspiration
C. Lysimeter	3. Infiltration
D. Hygrometer	4. Wind speed

Codes :

	A	B	C	D
(*a*)	4	3	1	2
(*b*)	3	4	1	2
(*c*)	4	3	2	1
(*d*)	3	4	2	1

Q. 4. If the life of a reservoir is determined by its capacity (C), volume of annual inflow into the reservoir (I) and concentration of sediment in the incoming flow (C_s). Life will be more if :

(*a*) C, I and C_s are high

(*b*) C and I are high but C_s is low

(*c*) C is high but I and C_s are low

(*d*) C, I and C_s are low.

Q. 5*. The probability that a 100-year flood is equalled or exceeded at least once in 100 years is :

(*a*) 99% (*b*) 64%

(*c*) 36% (*d*) 1%

Q. 6. A 3-hour storm on a small drainage basin produced rainfall intensities of 3.5 cm/hr, 4.2 cm/hr and 2.9 cm/hr in successive hours. If the surface runoff due to storm is 3 cm, then the value of ϕ-index will be :

(*a*) 2.212 cm/hr

(*b*) 2.331 cm/hr

(*c*) 2.412 cm/hr

(*d*) 2.533 cm/hr.

Q. 7. As a flood wave passes a given section of a river, the time of occurrence of the maximum stage and that of the maximum discharge will be such that :

(*a*) the maximum discharge passes down before the maximum stage is attained

(*b*) the maximum stage is attained before the maximum discharge passes down

(*c*) the two events occur simultaneously

(*d*) no specific sequence would be universally assignable.

Q. 8. In a curved reach of a meandering river, both deposition of sediments and erosion of bank occur. Which one of the following statements is true in this regard ?

(*a*) Deposition of sediments occurs in the inner bank while the outer bank is subjected to erosion

(*b*) Deposition of sediments occur in the outer bank while the inner bank is subjected to erosion

(*c*) In the direction of flow, the outer bank undergoes erosion first and the sediment is later deposited on the outer bank downstream

(*d*) In the direction of flow, the inner bank undergoes erosion first and the sediments are subsequently deposited on the inner bank downstream.

Q. 9. Gibb's module is a type of outlet which ensures :

(*a*) Constant discharge even if the water levels in the supply channel and water course fluctuate

(*b*) Variable discharge as per the need

(*c*) Constant discharge into the water course when the water levels in the supply channel vary

(*d*) Constant discharge for varying water levels in the water course for a given water level in the supply channel.

Q. 10. The Lacey's silt factor for a particular alluvium is 2.0. This alluvium would comprise :

(*a*) medium sand of size 0.5 mm (*b*) coarse sand of size 0.75 mm

(*c*) medium bajri of size 1.3 mm (*d*) coarse bajri of size 2.4 mm.

Q. 11. In a barrage on pervious foundation, sheet piles are provided both upstream and downstream of the barrage to reduce uplift pressure and to prevent piping. Which one of the following statements is true in this regard ?

(*a*) Compared to downstream sheet pile, the upstream sheet pile is more effective in reducing uplift and piping

(*b*) Compared to upstream sheet pile, the downstream sheet pile is more effective in reducing uplift and piping

(*c*) Downstream sheet pile is more effective in reducing uplift while the upstream sheet pile is more effective in reducing piping

(*d*) Upstream sheet pile is more effective in reducing uplift while the downstream sheet pile is more effective in reducing piping.

Q. 12. The following data pertains to a natural drain crossing an irrigation canal :

Item	*Canal data*	*Drainage data*
Flow (m^3/s)	5	500
Bed level (m)	120	116
Depth of flow (m)	0.8	10

Which one of the following types of cross-drainage should be recommended in this case ?

(*a*) Aqueduct (*b*) Syphon aqueduct

(*c*) Syphon (*d*) Super passage

Q. 13. Consider the following statements :

The downstream impervious floor of concrete for a barrage has ruptured. This can be due to :

1. insufficient length of upstream impervious floor
2. insufficient length of downstream impervious floor
3. insufficient depth of downstream pile
4. choking of inverted filter.

Which of these statements are correct ?

(*a*) 1 and 2 (*b*) 1 and 4

(*c*) 2, 3 and 4 (*d*) 1, 2, 3 and 4.

Q. 14*. Ten m^3/s of water is diverted to a 32 hectare field for 4 hours. Soil proving after irrigation showed that 0.3 m of water has been stored in the root zone. Water application efficiency in this case would be :

(*a*) 96% (*b*) 66.67%

(*c*) 48% (*d*) 24%.

Q. 15. Which one of the following statements is correct ?
In a river, silt excludes and silt ejector are constructed
(*a*) at a location after the head regulator and at the head of the canal, respectively
(*b*) at the head of the canal and at a location after the head regulator, respectively
(*c*) at the same location
(*d*) at specific locations depending upon diverse factors and their locations do not follow a set pattern.

ANSWERS

1. (*b*)	**2.** (*d*)	**3.** (*c*)	**4.** (*b*)	**5.** (*b*)
6. (*d*)	**7.** (*c*)	**8.** (*a*)	**9.** (*a*)	**10.** (*c*)
11. (*d*)	**12.** (*b*)	**13.** (*d*)	**14.** (*b*)	**15.** (*b*)

Hints for Solving Starred Questions of the year 2001

Q. 5. See Eqn. (7.135) and compute :

$$p = \frac{1}{T} = \frac{1}{100} = 0.01$$

Probability of non-occurrence of this flood in next 100 yrs

$$= p_{o,n} = q^n = (1-p)^n$$
$$= (1-0.01)^{100} = 0.36$$

Probability of occurrence of this flood at least once in 100 successive yrs

$$= R = 1 - q^n$$
$$= 1 - 0.36 = 0.64 \qquad i.e.\ 64\% \quad \textbf{Ans.}$$

Q. 14. Water depth applied to fields

$$= \frac{10\,\frac{m^3}{s} \times (4 \times 60 \times 60\ s)}{(32 \times 10^4\ m^2)} = 0.45\ m$$

Water depth actually stored in root zone. = 0.3 m

$$\eta_a = \frac{0.3\ m}{0.45\ m} = \frac{2}{3} = \mathbf{66.67\%} \quad \textbf{Ans.}$$

Year 2002 (*Engineering Services*)

Q1. The following items relate to hydrologic requirements in a hydropower system. Match List I with List II and select the correct answer using the codes given below the lists :

	List I		*List II*
A	Stream flow in order of magnitude is plotted as ordinate and percent of time as abscissa	1	Mass curve
B	Cumulative value of stream flow is plotted against time for whole period record	2	Flow duration curve
C	Peat of available power against percent of time	3	Hydrograph
D	Plot of flow in stream against time at specific interval	4	Power duration curve

Codes :

	A	B	C	D
(a)	2	3	4	1
(b)	4	1	2	3
(c)	2	1	4	3
(d)	4	3	2	1

Q2. Consider the following statements :

1. An ephemeral stream is one which has a base flow contribution.
2. Flow characteristics of a stream depend upon rainfall and catchment characteristics, and also the climatic factors which influence evapotranspiration.
3. Sequent Peak Algorithm is used for estimating run off from rainfall.

Which of these statements is/are correct ?

(a) 1, 2 and 3 (b) 1 and 3

(c) 2 and 3 (d) 2 clone.

Q3. Match List I with List II and select the correct answer using the codes given below the lists :

List I	*List II*
A ϕ index	1. Used for measurement of evapo-transpiration for given vegetation
B Lysimeter	2. Used for flow measurement
C Dilution technique	3. Average rainfall above which the rainfall volume is equal to the run-off volume
D Snydel's equation	4. Relates the basin lag to the basin characteristics

Codes :

	A	B	C	D
(a)	3	1	2	4
(b)	4	2	1	3
(c)	3	2	1	4
(d)	4	1	2	3

Q4. A 6-hour rainstorm with hourly intensities of 7, 18, 25, 17, 11 and 3 mm/hour produced a run off of 39 mm. Then, the ϕ_{index} is

(a) 3 mm/hour

(b) 7 mm/hour

(c) 8 mm/hour

(d) 10 mm/hour.

Q5.* In a water-shed, four rain gauges I, II, III and IV are installed. The depths of normal annual rainfall at these stations are 60, 75, 80 and 100 cm, respectively. The rain gauge at station II went out of order during a particular year. The annual rainfall for this

year, recorded at the remaining three stations was 90, 60 and 70 cm. The rainfall at station III can be considered as

(*a*) 60 cm (*b*) 70 cm

(*c*) 80 cm (*d*) 120 cm

Q6. The unit hydrograph theory is based on the assumption of

(*a*) Nonlinear response and time invariance

(*b*) Linear response and non-linear time variance

(*c*) Time invariance and linear response

(*d*) Non-linear response and non-linear time variance.

Q7. An effective storage of a flood control reservoir is

(*a*) the amount of water which can be supplied from it in a particular interval of time

(*b*) the storage between the minimum and maximum reservoir levels under ordinary operating conditions

(*c*) the useful storage plus the surcharge storage less the valley storage

(*d*) the storage volume of flood water above the maximum reservoir level.

Q8. The lenman's evapo-transpiration equation is based on

(*a*) water budget method

(*b*) energy balance method

(*c*) mass transfer method

(*d*) energy balance and mass transfer approach.

Q9. The performance of a well is measured by its :

(*a*) specific capacity (*b*) specific yield

(*c*) storage coefficient (*d*) permeability coefficient.

Q10. Regional hydrological cycle is shown in the figure below (Fig. 32.2)

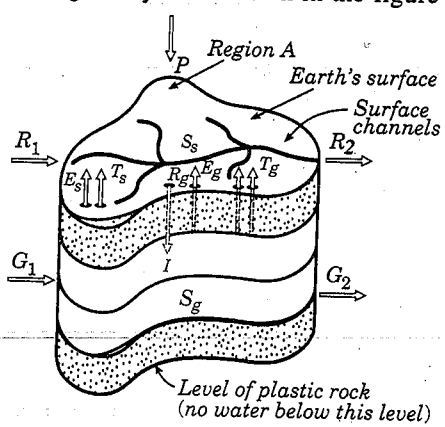


Fig. 32.2.

Identify the correct hydrologic budget equation

(*a*) $P + R_1 - R_2 + R_g - E_s - T_s - I = \Delta S_s$

(*b*) $I + G_1 - G_2 - R_g - E_g - T_g = \Delta S_g$

(*c*) $P-(R_2-R_1)-(E_s+E_g)-(T_s+T_g)-(G_2-G_1)=\Delta(S_s+S_g)$

(*d*) $P-R-G-E-T=\Delta S_s$.

Q11. A two-hour storm hydrograph has 5 units of direct run-off. The two-hour unit hydrograph for this storm can be obtained by dividing the ordinates of the storm hydrograph by

(*a*) 2

(*b*) $\frac{2}{5}$

(*c*) 5

(*d*) $\frac{5}{2}$.

Q12. The slope area method is extensively used in

(*a*) development of rating curve

(*b*) estimation of flood discharge based on high water marks

(*c*) cases where shifting control exists

(*d*) cases where back-water effect is present.

Q13. Consider the following :

Garret's diagram for the design of irrigation channel is based on

1. Kennedy's theory
2. Lacey's theory
3. Kutter's formula
4. Manning's formula.

Which one of these are correct ?

(*a*) 1 and 3 (*b*) 1 and 4

(*c*) 2 and 3 (*d*) 2 and 4.

Q14. The total number of dependent equations that form the Lacey's regime theory is

(*a*) 2 (*b*) 3

(*c*) 4 (*d*) 6.

Q15. Balanced depth of cutting of canal is

(*a*) half the total depth of a canal

(*b*) half of full supply depth

(*c*) the maximum cut that an excavator can take

(*d*) where volume of cutting is equal to volume of filling.

Q16. At a certain point in the floor of weir, the uplift pressure head due to seepage in 4.5 m. If the relative density of concrete is 2.5, the minimum thickness of floor required at this point to counteract the uplift pressure is

(*a*) 1 m (*b*) 2 m

(*c*) 3 m (*d*) 4 m

Q17. A constant angle arch dam when compared to a constant radius arch dam utilizes concrete quantity of about

(*a*) 33% (*b*) 43%

(*c*) 73% (*d*) 143%

Q18. Drowning ratio of canal outlet is the ratio of

(*a*) rate of change of discharge of outlet to that of distributary

(*b*) water depth above outlet crest to the full supply depth of the channel

(*c*) water depth above the crest on downstream to that on upstream of outlet

(*d*) rate of change of discharge of an outlet to the rate of change of water level of the channel.

Q19. A canal was designed to supply the irrigation needs of 1000 *ha* of land growing rice of 140 days base period and having a delta of 130 cm. If the canal water is used to irrigate wheat of base period 119 days and having a delta of of 50 cm, the area that can be irrigated is ;

(*a*) 452 ha (*b*) 904 ha

(*c*) 1105 ha (*d*) 2210 ha

Q20. A canal fall is a control structure

(*a*) located at a place where the country slope is flatter than the canal bed slope

(*b*) located most economically where the depth of cutting is less than the balancing depth

(*c*) the location of which is independent of the command to be served

(*d*) designed to secure raising of water surface on its upstream.

Q21. Match List I (Equation) with List II (Applicability or principle of equation) and select the correct answer using the codes given below the lists :

List I	*List II*
A Theim's equation	1 is based on energy conservation principle
B Dupuit's assumption	2 is based on mass conservation principle
C Bernoullis equation	3 is applicable to steady flow towards a well in confined aquifer
D Continuity equation	4 is applicable to steady flow in an unconfined aquifer

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	3	4	2	1
(*c*)	4	3	1	2
(*d*)	3	4	1	2

ANSWERS

1. (*c*)	**2.** (*d*)	**3.** (*a*)	**4.** (*c*)	**5.** (*b*)	**6.** (*c*)	**7.** (*c*)
8. (*d*)	**9.** (*a*)	**10.** (*c*)	**11.** (*c*)	**12.** (*b*)	**13.** (*a*)	**14.** (*d*)
15. (*d*)	**16.** (*c*)	**17.** (*b*)	**18.** (*c*)	**19.** (*d*)	**20.** (*b*)	**21.** (*c*)

Hints for Solving Sttared Questions of the Year 2002

Q5. Station I, N_1 = Annual Av. rainfall = 60 cm

Station II, N_2 (N_x) = Annual Av. rainfall = 75 cm

Station III, N_3 = Annual Av. rainfall = 80 cm.

Station IV, N_4 = Annual Av. rainfall = 100 cm

$P_1 = 90$ cm $\qquad P_3 = 60$ cm

$P_4 = 70$ cm $\qquad P_2 = ?$

Since the value of anyone av. annuai ppt. of known stations, *i.e* N_1, N_3 and N_4 or differs from N_2 (*i.e.* N_x) by more than 10%, is applicables as :

$$\therefore \qquad P_x = P_2 = \frac{N_x}{3}\left[\frac{P_1}{N_1} + \frac{P_3}{N_3} + \frac{P_4}{N_4}\right]$$

$$= \frac{75}{3}\left[\frac{90}{60} + \frac{60}{80} + \frac{70}{100}\right] = 25\,[1.5 + 0.75 + 0.7]$$

$$= 25 \times 2.95 = 73.75 \text{ cm} \quad \textbf{Ans.}$$

The nearest choice available in question is 70 cm, at choice (*b*), which is marked as the correct choice. **Ans.**

Q19. Design discharge for canal

Rice Area to be irrigated = 1000 ha, B for Rice = 140 days and Δ for rice = 130 cm

Duty (D) for rice

$$= \frac{864\,B}{\Delta} = \frac{864 \times 140}{130}$$

$$= 930.46 \text{ ha/cumec}$$

Discharge reqd for above duty

$$= \frac{1000}{930.46} \text{ cumec} = \textbf{1.0747 cumec}$$

When the above discharge of canal is used for irrigating wheat crop of

$B = 119$ days $\qquad \Delta = 50$ cm.

Duty for wheat $(D) = \dfrac{864\,B}{\Delta} = \dfrac{864 \times 119}{50} = 2056.32$ ha/cumec.

The ha which can be irrigated by 1.0747 cumec.

$$= 2056.32 \times 1.0747 \text{ ha} = \textbf{2210 ha}$$

Hence, choice at (*d*) is correct. **Ans.**

Year 2003 (*Engineering Services*)

Q1. The worst condition of uplift on the floor of a siphon aqueduct occurs when there is

(*a*) high flood flow in the drainage with canal dry

(*b*) full supply flow in the canal with drainage dry

(*c*) high flood flow in the drainage with canal running full

(*d*) water is at drainage bed and canal is dry.

Q2. A check dam is a

(*a*) flood control structure (*b*) soil conservation structure

(*c*) river training structure (*d*) water storage structure.

Q3. Match List I (Well Hydraulics Parameters) with List II (Definition) and select the correct answer using the codes given below the Lists :

List I (Well Hydraulics Parameters)		*List II (Definition)*	
A	Specific yield.	1	Discharge per unit draw down of well.
B	Safe yield	2	Same as specific retention.
C	Specific capacity	3	Measure of water that can be removed by pumping.
D	Field capacity	4	Limit of withdrawal from well without depletion of the aquifier
		5	Water bearing capacity of aquifer.

Codes :

	A	B	C	D
(*a*)	4	3	2	2
(*b*)	3	4	1	2
(*c*)	4	3	1	2
(*d*)	3	4	2	5

Q4. Consider the following statements :

A well development

1. involves reversal of flow through the well screen
2. increases permeability towards the well
3. decreases permeability towards the well
4. is continued till sand/silt free water is pumped out

Which of these statements is/are correct ?

(*a*) 1, 3 and 4 (*b*) 1, 2 and 4

(*c*) 3 only (*d*) 1 and 4.

Q5. Leaching is a process

(*a*) by which alkali salts present in the soil are dissolved and drained away

(*b*) by which alkali salts in soil come up with water

(*c*) of draining excess water of irrigation

(*d*) which controls water-logging.

Q6. Directions

The following items consist of two statements, one labelled as the 'Assertion A' and the other as 'Reason (R)'. You are to examine these two statements carefully and select the answers to these items using the codes given below :

Codes :

(*a*) Both A and R are individually true and R is the correct explanation of A

(*b*) Both A and R are individually true but R is not the correct explaination of A

(*c*) A is true but R is false

(*d*) A is false but R is true.

(i) *Assertion (A)* : With tile lining of canals, permissible velocity of flow is lower than that with concrete lining.

Reason (R) : The surface of the lining becomes rough due to loss of surface material with high velocity.

(ii) *Assertion (A)* : Canal escape serves as a safety valve for a canal.

Reason (R) : Canal escape discharges the excess water in the parent canal due to sudden closure of outlets by farmers.

Q7. Which one of the following phenomena in a pipe flow is termed as water hammer ?

(*a*) The sudden rise of pressure in a long pipe due to sudden closure of valve

(*b*) the rise of a pressure in a pipe flow due to gradual closure of valve

(*c*) The rise of negative pressure

(*d*) The zero pressure in a pipe flow.

Q8. Flow duration curve is a plot of

(*a*) Flow against its time of occurrence in chronological order

(*b*) Flow in ascending order against percentage time in chronological order

(*c*) Flow that equalled or exceeded against percentage time

(*d*) Flow against duration of time for which it is sustained.

Q9*. Pondage in a hydropower station is defined as :

(*a*) impounding of considerable amount of excess water during seasons of surplus flow

(*b*) A regulating body of water in the form of relatively small amount of run-off to regulat flow variation in daily or weekly power requirements

(*c*) Excess run off to last for years

(*d*) Excess run off for a few hours only.

Q10. The specific speed of a turbine under a head of 150 m to develop 2000 HP while running at 300 rpm is

(*a*) 10—35 (*b*) 35—60

(*c*) 60—300 (*d*) 300—1000.

Q11. The moving average of annual precipitation record is carried out to determine

(*a*) Trend (*b*) Annual mean

(*c*) Extreme annual variation (*d*) Extreme seasonal variation.

Q12. Match List I (Hydrological Terms) with List II (Relationship/Nature of Curve) and select the correct answer using the codes given below the Lists :

List I (Hydrological Terms)	*List II (Relationship/Nature of Curve)*
A Theissen Polygon	1 Average depth of rainfall over an area
B Mass Curve	2 Relationship of rainfall intensity and time
C Hyetograph	3 Relationship of accumulated rainfall and time
D DAD curve	4 Relationship of river run-off and time
	5 Always a falling curve.

Codes :	A	B	C	D
(*a*)	1	3	2	5
(*b*)	1	5	3	2
(*c*)	4	3	2	5
(*d*)	4	5	3	2

Q13. Match List I (Type of Precipitation) with List II (Principal Causes) and select the correct answer using the codes given below the Lists :

List I (Type of Precipation)	*List II (Principal Causes)*
A Convective	1 Atmospheric disturbance
B Cyclonic	2 Mountain barrier
C Frontal	3 Pressure difference
D Orographic	4 Temperature difference
	5 Warm and cold air masses

Codes :

	A	B	C	D
(*a*)	1	4	5	2
(*b*)	4	3	5	2
(*c*)	1	4	2	5
(*d*)	4	3	2	5

Q14. Which one of the following characteristics describes a watershed system in system's parlance ?

(*a*) Linear (*b*) Non-linear

(*c*) Linear and time-invariant (*d*) Non-linear and time-variant.

Q15. A 6-hour storm with hourly intensities of 7, 18, 25, 12, 10 and 3 mm per hour produced a run-off of 33 mm. Then the ϕ_{index} is

(*a*) 7 mm/h (*b*) 3 mm/h (*c*) 10 mm/h (*d*) 8 mm/h.

Q16. A catchment area of 90 hectares has a run-off coefficient of 0.4. A storm of duration larger than the time of concentration of the catchment and of intensity 4.5 cm/hr creates a peak discharge rate of

(*a*) 11.3 m^3/s (*b*) 0.45 m^3/s (*c*) 450 m^3/s (*d*) 4.5 m^3/s.

Q17. Consider the following statements :

1. Time-area histogram method aims at developing an IUH
2. Isochrone is a line joining equal rainfall on a map
3. Linear reservoir is a reservoir having straight boundaries
4. Linear channel is a fictitious channel in which an inflow hydrograph passes through, with only translation and no attenuation.

Which of these statements are correct ?

(*a*) 1, 2 and 3 (*b*) 1 and 4

(*c*) 2, 3 and 4 (*d*) 1, 2, 3 and 4.

Q18. Match List I (Floods) with List II (Parameters) and select the correct answer using the codes given below the Lists :

List I (Floods)		*List II (Parameters)*	
A	Standard Project Flood (SPF)	1	Includes catastrophic floods
B	Maximum Probable Flood (MPF)	2	Includes floods of severe conditions
C	Design Flood	3	Peak flow obtained from observed data
D	Maximum Flood	4	Flood of desired recurrence interval

Codes :

	A	B	C	D
(*a*)	2	1	4	3
(*b*)	1	2	3	4
(*c*)	2	1	3	4
(*d*)	1	2	4	3

Q19. Which of the following pairs are correctly matched ?

1. Silt factor : Average size of silt particle.
2. Silt load : Volume of suspended sediments flowing with water in unit time.
3. Silt charge : Weight of silt per unit volume of water
4. Silt grade : Gradation between different silt particles

Select the correct answer using the codes given below :

Codes

(*a*) 1 and 4 (*b*) 3 and 4 (*c*) 2 and 3 (*d*) 1 and 2.

Q20. Consider the following statements :

The function of a cut-off in an earth dam is to

1. Reduce uplift pressures on the dam
2. Prevent undermining of foundation
3. Reduce loss of stored water
4. Support the dam

Which of these statements are correct ?

(*a*) 1 and 3 (*b*) 2 and 4 (*c*) 2 and 3 (*d*) 3 and 4.

Q21. Match List I (Theory) with List II (Propounded By) and select the correct answer using the codes given below the Lists :

List I (Theory)	*List II (Propounded By)*
A Exit gradient	1 G. Lacey
B Alluvial canal	2 L.K. Sherman
C Unit hydrograph	3 A.N. Khosla
D Boundary layer	4 C. Inglis
	5 T.V. Karman
	6 L. Prandtl

Codes :

	A	B	C	D
(*a*)	1	3	2	6
(*b*)	6	2	3	5
(*c*)	3	1	2	6
(*d*)	3	1	4	2

Q22. The ideal condition for energy dissipation in the design of spillways is the one when the tail water rating curve

(*a*) Lies above jump curve at all discharges

(*b*) Coincides with the jump rating curve at all discharges

(*c*) Lies below jump rating curve at all discharges

(*d*) Lies either above or below the jump rating curve depending upon discharge.

ANSWERS

1. (*d*)	**2.** (*b*)	**3.** (*b*)	**4.** (*b*)	**5.** (*a*)
6. (*i*)-(*a*), (*ii*)-(*a*)		**7.** (*a*)	**8.** (*c*)	**9.** (*b*)
10. (*a*)	**11.** (*a*)	**12.** (*a*)	**13.** (*b*)	**14.** (*c*)
15. (*d*)	**16.** (*d*)	**17.** (*b*)	**18.** (*a*)	**19.** (*c*)
20. (*c*)	**21.** (*c*)	**22.** (*b*)		

Hints for Solving Starred Quetions of the Year 2003

Q9. As pointed out in article 24.2.1, run-off river plants installed on main rivers or on diversion canals are usually provided with a small storage to meet the short term fluctuations of the water requirement of the plant. A balancing reservoir to store excess stream flow during periods of low consumption in the power house during off peak hours or days, and for its release during excess load periods, is essentially required in all such low head power development schemes. Such a balancing reservoir of small capacity, called a *pond*, is hence required near the power house.

In case of a power plant located on the river, the main weir itself provides on its upstream side the required pondage ; while in the case of a plant located on a diversion canal (power canal) the pond is required to be created at the end of the canal in the form of a **foreway reservoir**, from which the penstock pipes shall lead the water into the power house, as shown in Fig. 24.2 (*b*). No matter where the pond is located, its capacity has to be worked out considering the inflow and outflow patterns, just as in the case of

storage reservoirs. As a matter of fact, the storage reservoirs provide storage of water for long term requirements, whereas, the *pondage* takes care of the hour to hour fluctuations over a day, or occassionally, day to day fluctuations over a weekly cycle. Provision of pondage ensures that the desired instantaneous rate of flow into the turbines under day to day changing conditions of power load as well as of inflow.

Short term fluctuations may arise due to the following reasons :

(*i*) The power consumption and hence the load on turbines may suddenly increase or decrease, needing in a few seconds either excess or lesser water, respectively. This change in water demand of the power house can be immediately met by the stored water available in the pond created for this purpose. This pond would provide extra water when needed, and retain excess flow when not needed in the plant.

(*ii*) The power load and, thus, the water demand of the power house, may be steady, but the supply of water may undergo a change. In natural streams, where some other power house is located on the upstream, the available flow may change depending upon the quantity of water released from the upstream power house.

Similarly, if the power house in question is located on a diversion channel, the inflow into the power house may get affected due to some breach(es) that may occur in the said diversion canal.

(*iii*) The water demand of turbines and the natural flow (supply) may change from time to time.

The pondage, in some cases, may also be provided for weekly demands. In most of the advanced countries, the free weekends result in decreased demand of power, reducing the water need of the given power house. The inflow during this period could be ponded for use during week days.

Year 2004 (*Engineering Services*)

Q1.* Which one of the following defines aridity index (*AI*) ?

(*a*) $AI = \dfrac{PET - AET}{PET} \times 100$ (*b*) $AI = \dfrac{PET}{AET} \times 100$

(*c*) $AI = \dfrac{AET}{PET} \times 100$ (*d*) $AI = \dfrac{AET - PET}{AET} \times 100$

Q2. In Snyder's method of synthetic unit hydrograph development, basin lag is taken as

(*a*) the time interval between centroid of the rainfall excess and surface runoff
(*b*) the time interval from mid point of the unit rainfall excess to the peak of the unit hydrograph
(*c*) independent of rainfall duration
(*d*) independent of catchment characteristics.

Q3. Which one of the following is the correct sequence in the increasing order of the Froude number of flow assumed by the bed form of an alluvial stream with movable bed material ?

(*a*) Ripple — Plane bed — Dune — Plane bed — Antidune
(*b*) Dune — Ripple — Plane bed — Antidune — Plane bed
(*c*) Plane bed — Ripple — Dune — Plane bed — Antidune
(*d*) Plane bed — Ripple — Antidune — Dune — Plane bed.

Q4. A bridge has an expected life of 50 years and is designed for a flood magnitude of return period 100 years. What is the risk associated with this hydrologic design ?

(*a*) $1 - (0.99)^{50}$ (*b*) $(0.5)^{50}$ (*c*) $(0.99)^{50}$ (*d*) $(0.99)^{100}$

Q5. Match List-I with List-II and select the correct answer using the codes given below the lists :

List I		*List II*	
A	Rising limb of a hydrograph.	1	Depends on intensity of rainfall
B	Falling limb of a hydrgraph	2	Function of total channel length
C	Peak rate of flow	3	Function of catchment slope
D	Drainage density	4	Function of storage characteristics

Codes :

	A	B	C	D
(*a*)	3	4	1	2
(*b*)	1	4	3	2
(*c*)	3	2	1	4
(*d*)	1	2	3	4

Q6. Match List-I with List-II and select the correct answer using the codes given below the lists :

List I		*List II*	
A	Unit hydrograph	1	Design flood
B	Synthetic unit hydrograph	2	Pemeability
C	Darcy's law	3	Ungauged basin
D	Rational method	4	1 cm runoff

Codes :

	A	B	C	D
(*a*)	2	3	4	1
(*b*)	2	1	4	3
(*c*)	4	3	2	1
(*d*)	4	1	2	3

Q7. Which one of the following statements is correct in respect of the two important aspects of flood forecast — (1) reliability of the forecast, and (2) the time available in between the forecast and the occurrence of flood ?

(*a*) Meteorological forecast is least reliable and time available is also the least

(*b*) Hydrological forecast is most reliable but the time available is the least

(*c*) River forecast is least reliable and the time available is the maximum

(*d*) River forecast is most reliable but the time available is the least.

Q8. The delta for a crop having base period 120 days is 70 cm. What is the duty ?

(*a*) 2480 hectare/cumec (*b*) 1481 hectare/cumec

(*c*) 148 hectare/cumec (*d*) 1.481 hectare/cumec.

Q9. For a culturable command area of 1000 hectare with intensity of irrigation of

50%, the duty on field for a certain crop is 2000 hectare/cumec. What is the discharge required at head of water course with 25% losses of water ?

(*a*) 3/16 cumec (*b*) 1/4 cumec
(*c*) 1/3 cumec (*d*) 1/2 cumec.

Q10. What is the moisture depth available for evapotranspiration in root zone of 1 m depth soil, if dry weight of soil is 1.5 gm/cc, field capacity is 30% and permanent wilting point is 10% ?

(*a*) 450 mm (*b*) 300 mm
(*c*) 200 mm (*d*) 150 mm.

Q11. Which one of the following correctly defines aquiclude ?

(*a*) A saturated formation of earth material which not only stores water but also yields it in sufficient quantity

(*b*) A formation through which only seepage is possible and thus the yield is insignificant compared to an aquifer

(*c*) A geological formation which is neither porous nor permeable

(*d*) A geological formation which is essentially impermeable to the flow of water.

Q12. What is the regime scour depth for a channel in soil with silt factor of unity and carrying 8 m^2/s of discharge intensity in accordance with Lacey's regime theory ?

(*a*) 3.6 m (*b*) 4 m
(*c*) 5.4 m (*d*) 25.6 m.

Q13. Which one of the following equations represents the downstream curve of the "Ogee" spillway (where x and y are the co-ordinates of the crest profile measured from the apex of the crest and H is the design head) ?

(*a*) $x^{1.85} = 2H^{0.85}y$ (*b*) $x = 2H^{1.85}\, y^{0.85}$
(*c*) $x^{0.85} = 2H^{1.85}y$ (*d*) $x = 2H^{0.85}y^{1.85}$.

Q14.* Clark's method aims at which one of the following :

(*a*) Developing an IUH due to an instantaneous rainfall excess over a catchment

(*b*) Developing stage-discharge relationship

(*c*) Measurement of infiltration

(*d*) Flood routing through channels.

Q15.* For a saddle siphon, the maximum operative head is 6.25 m. The width and height of the throat of the siphon are 4 m and 2 m respectively. The coefficient of discharge is 0.90. How many units are required to pass a flood of 300 cumec ? (Take $g = 10$ m/s^2)

(*a*) One (*b*) Two
(*c*) Three (*d*) Four.

ANSWERS

1. (*a*)	**2.** (*b*)	**3.** (*c*)	**4.** (*a*)	**5.** (*a*)
6. (*c*)	**7.** (*d*)	**8.** (*b*)	**9.** (*c*)	**10.** (*b*)
11. (*d*)	**12.** (*c*)	**13.** (*a*)	**14.** (*a*)	**15.** (*d*).

Hints for Solving Sterred Questions of the Year 2004

Q1. Aridity Index, as its name implies, is an indicator of the dryness (aridness) of a soil/crop, due to which the plants do not get their full required quantity of water upto the field capacity. The max. available moisture [*i.e.* Field capacity *mc* – Wilting point *mc*], thus, does not become available to the crops, which reduces their actual evapotranspiration (*AET*) in comparison to the potential (*i.e.* maximum) evapotranspiration (*PAT*). The deficiency in evapotranspiration [*i.e.* (*PAT* – *AET*)] when expressed as the ratio with *PAT* is, hence, defined as the *Aridity Index*. Thus,

$$\text{Aridity Index} = AI = \frac{PAT - AET}{PAT}\ (\%)$$

Thus, if *PAT* is 80 mm and *AET* is 20 mm, then, naturally, the crops will be facing $\frac{80-20}{80} = 75\%$ dry conditions, since the aridity index is 75%.

Hence, choice (*a*) is correct. **Ans.**

Q14. Clark's method is one of those methods, which are used for computing an *Instantaneous Unit Hydrograph (IUH)*. Since an *IUH* is a concept of advanced Hydrology, we have not discussed it in our text. However, an IUH is explained below in brief.

Instantaneous Unit Hydrograph (IUH). The ordinary *UH*, as you know by now, is obtained from an excess rainfall (*ER*) of 1 cm magnitude uniformally falling during a time period of *T* hour, with an intensity equal to $\frac{1}{T}$ cm/h. If this rain duration (*T*) is progressively decreased, its intensity will go on increasing, and the resulting unit hydrograph will become more skewed, as shown in Fig. 32.3 by hydrograph marked 2. The hydrographs will go on becoming more skewed with progressively reduced value of *T*, as shown by unit graphs marked 3 and 4. A more skewed but certainly of finite shape unit hydrograph will finally be obtained, when the rainfall duration *T* is made infinitely small ($T \to 0$). This limiting case of a unit hydrograph produced by a rainfall of 1 cm in zero hr time, is called an *instantaneous unit hydrograph (IUH)*. Of course, this is only a theoretical concept, because 1 cm of rain cannot fall in zero hr time on any catchment, *but is useful because such a unit graph represents the watershed response to any amount of rainfall excess without reference to the duration of the*

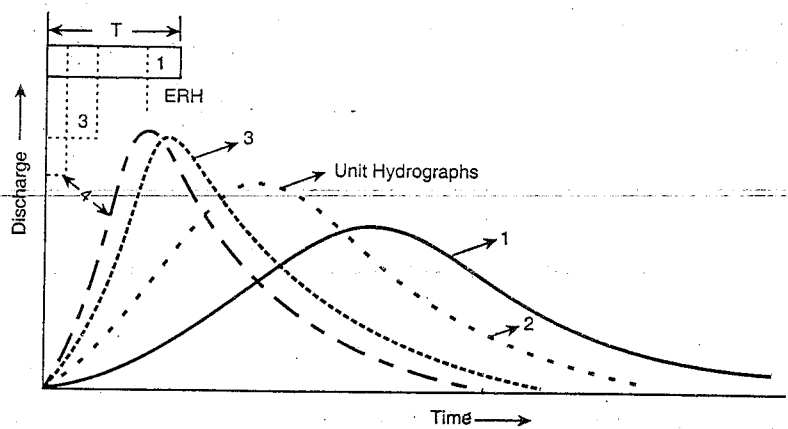


Fig. 32.3. Some Typical Unit Hydrographs.

rainfall. An *IUH*, being independent of rainfall characteristics, is indicative of the storage characteristics of a catchment, and is eminently suitable for theoretical analysis of effective rainfall-runoff relationship of a catchment.

The ideal shape of an IUH resembles that of a single-peaked direct runoff hydrograph (*DRH*) with a finite base width ; yet however, an *IUH* can have negative and undulating ordinates.

The *IUH* for a catchment can be derived from a given *ERH* (Excess rain hydrograph) and *DRH* (Direct runff hydrograph) by several methods. Say for example, the ordinate of an *IUH* at any time t has been shown to be equal to the slope at time t of the *S*-curve caused by an excess rain of intensity 1 cm/h (*i.e. S*-curve derived from a *UH* of 1-h duration). The *IUH* so obtained is, infact, an approximate one, because the slope of *S*-curve is difficult to be measured accurately.

Mathematical solutions evolved for determining *IUH* include. Laplace transform (chow 1964), Fourier transform (Black and others, 1971) and *Z*-transform (Bree 1978). These methods involve mathematical modelling related to watershed geomorphology, harmonic analysis, etc. which are beyond the scope of this book.

A comparatively easier method for developing an *IUH* has been evolved by Clark, and is hence known as the *Clark's method* or **Time-area histogram method.** This method aims at developing an *IUH* due to an instantaneous excess rainfall over a catchment. It is assumed that the rainfall excess first undergoes pure *translation* (channel movement) in the catchment, and then *attenuation* (reduced peak and elongated time base) as in reservoir routing. The translation is computed upto the catchment outlet by a travel time-area histogram (by ignoring the storage of the catchment) ; while the attenuation is computed by routing the result of the above, through a hypothetically available linear reservoir at the catchment outlet, to account for the storage in the catchment. A detailed description of the computations is beyond the scope of this book.

Q15. From Eq. (21.13), discharge through a saddle syphon is given as ;

$$Q = C_d \cdot A \cdot \sqrt{2gH_1}$$

where $C_d = 0.9$

A = Area of flow width × Ht. of Throat

$= 4 \text{ m} \times 2 \text{ m} = 8 \text{ m}^2$ (given)

$g = 10 \text{ m}^3/\text{s}$ (given)

$H_1 = 6.25$ m (given)

$$\therefore \quad Q = 0.9 \times 8\sqrt{2 \times 10 \times 6.25} = \mathbf{80.5 \text{ m}^3/\text{s}}.$$

Total discharge = 300 cumec (given)

No. of units of saddle syphon $= \dfrac{300}{80.5} = 3.73$; Say 4.

Hence, choice (*d*) is correct. **Ans.**

—33

Objective Questions of AMIE Exams (New Scheme)

Year 1993 (*Summer*) **Exam**

Q. Complete the sentences

(*i*) If the duty of water for a crop of base period 120 days is 1400 hec/cumec, then the delta (Δ) for the crop is

(*a*) 741 mm ; (*b*) 843 mm ;

(*c*) 949 mm ; (*d*) 1056 mm.

(*ii*) According to Khosla's theory the exit gradient is given by

(*A*) $\frac{d}{H}\frac{1}{\pi\lambda}$; (*B*) $\frac{H}{d}\frac{1}{\pi\sqrt{\lambda}}$;

(*C*) $\frac{H}{d}\pi\sqrt{\lambda}$ (*D*) $\frac{d}{H}\pi\lambda$.

(*iii*) If the dominant discharge of an alluvial river is 1800 cumecs, the waterway for a bridge on the river should be

(*A*) 101.5 m ; (*B*) 151.5 m ;

(*C*) 201.5 m ; (*D*) 251.5 m.

(*iv*) Slite excluder is provided

(*A*) in the canal on the downstream side of the head regulator

(*B*) in the river on the downstream side of the barrage ;

(*C*) in the river far away from the weir on the downstream side

(*D*) in the river adjacent to the head regulator.

(*v*) According to Dicken's formula, the peak flood discharge is proportional to

(*A*) area ; (*B*) $(\text{area})^{1/2}$;

(*C*) $(\text{area})^{2/3}$; (*D*) $(\text{area})^{3/4}$.

(*vi*) The head due to sudden expansion is expressed by

(*A*) $\frac{v_1^2 - v_2^2}{2g}$; (*B*) $\frac{(v_1 - v_2)^2}{g}$;

(*C*) $\frac{(v_1 - v_2)^2}{2g}$; (*D*) $\left(\frac{v_1 - v_2}{2g}\right)^2$.

(*vii*)* A rectangular channel of 4 m width carries water at the rate of 8 cumecs under critical conditions. Specific energy for the flow is...

(*viii*)* In a syphon aqueduct the most severe condition of uplift on the floor occurs when

(*A*) Canal runs full drain is dry but the water table is at stream bed ;

(*B*) Canal is dry and drain is at high flood level ;

(*C*) Canal runs dry and the drain also runs dry

(*D*) Both the canal and drain run full.

(*ix*)* If the conjugate depths for a horizontal rectangular channel 4 m wide are 0.2 m and 1 m the discharge in the channel is — m^3/s.

(*x*) If y_1 and y_2 are the depths of flow before and after the hydraulic jump in a rectangular channel then the loss of energy is given by

(*A*) $\frac{(y_2 - y_1)^3}{4y_1 y_2}$; (*B*) $\frac{(y_2 - y_1)^2}{4y_1 y_2}$;

(*C*) $\frac{(y_2 - y_1)^3}{y_1 y_2}$; (*D*) $\frac{(y_2 - y_1)^2}{y_1 y_2}$.

Answers to Questions of 1993 (*Summer*) Exam

(*i*)–(*a*)	(*ii*)–*B*	(*iii*)–C
(*iv*)–*D*	(*v*)–*D*	(*vi*)–*A*
(*vii*)–1.11	(*viii*)–*B*	(*ix*)–4.34
(*x*)–*A*		

Hints for Solving Starred Questions of 1993 (*Summer*) Exam.

Q. (*vii*) $B = 4$ m, $Q = 8$ cumecs at critical depth

q at critical depth $= \frac{8}{4} = 2\ m^3/s$

But $$y_c = \sqrt[3]{\frac{q^2}{g}} = \sqrt[3]{\frac{2^2}{9.81}} = 0.74\ m$$

Also, $$\frac{Q}{A} = v_c = \frac{8}{4 \times 0.74} = 2.7\ m/s$$

$$E_{fc} = y_c + \frac{v_c^2}{2g} = 0.74 + \frac{(2.7)^2}{2 \times 9.81} = \mathbf{1.11\ m} \quad \textbf{Ans.}$$

Q. (*ix*) $y_1 = 0.2$ m ; $y_2 = 1$ m

using eq. (10.6), we have,

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{g \cdot y_1}}$$

or $$1 = -\frac{0.2}{2} + \sqrt{\frac{0.2^2}{4} + \frac{2.q^2}{9.81 \times 0.2}}$$

or $$1 + 0.1 = \sqrt{0.01 + \frac{q^2}{0.981}}$$

$$1.1^2 = 1.21 = 0.01 + \frac{q^2}{0.981}$$

or $$q = \sqrt{1.20 \times 0.981} = 1.085$$

$$Q = q.\, B = 1.085 \times 4 = \mathbf{4.34\ m^3/s} \quad \textbf{Ans.}$$

Year 1993 (*Winter*) Exam

Q. (*A*) Fill in the blanks :

(*i*) Where water must be raised by pumps or other means to irrigate crops is called ..... irrigation.

(*ii*) Soil in which the groundwater table rises so high as to affect the productivity of the soil due to air circulation to the plant root zone is called .....

(*iii*) An irrigation outlet whose discharge is independent of water levels in the distributary and the water course and which ensures constant supply within certain working limits is called a .....

(*iv*) The scour depth for a channel in fine silt is ..... than in coarse silt.

(*v*) Excess flood over a concrete dam is released through the .....

(*vi*) The loss of water from plant surfaces and utilised in its growth is known as .....

(*B*) Choose the correct statement for the following :

(*i*) The bed slope of a canal off taking from a barrage is kept
- (*a*) steeper than the bed slope of the river
- (*b*) milder than the bed slope of the river
- (*c*) same as that of the river
- (*d*) none of the above

(*ii*) For the same height and top width, the width at the bottom of the earth dam is
- (*a*) less than that of a concrete dam
- (*b*) same as that of a concrete dam
- (*c*) more than that of a concrete dam
- (*d*) none of the above

(*iii*) For a given discharge, the afflux for a pipe culvert will be
- (*a*) the same as for a large opening
- (*b*) independent of the size of the opening
- (*c*) more than that for a large opening
- (*d*) none of the above

(*iv*) Lining of a canal is required when
- (*a*) the water table is above the full supply level of canal
- (*b*) the water table is below the bed of the canal
- (*c*) there is a natural impervious stratum below the bed
- (*d*) none of the above

(*v*) In a deflecting spur, the angle between the axis of the spur and the bank on the upstream side is

(*a*) acute

(*b*) obtuse

(*c*) same as on the downstream side

(*d*) one of these

(*vi*) Khosla's theory of indpendent variables is applicable to structure founded on

(*a*) rock (*b*) clay

(*c*) sands (*d*) concrete

(*vii*) A cross regulator is provided on a main canal

(*a*) to minimise the amount of silt entering the branch canal

(*b*) to let maximum silt be carried into the branch canal

(*c*) for no specific purpose

(*d*) to carry the canal across the drain

(*C*) Match the sentences of Column A to that in Column -B :

	Column-A	*Column-B,*
(*i*)	(*a*) In storage irrigation	(*A*) a barrage or a weir is constructed to divert water directly into the canal
	(*b*) In direct irrigation	(*B*) water is supplied to individual trees as in orchards
	(*c*) In sprinkler irrigation	(*C*) a dam is constructed to store water during the monsoons and supply water to canals during non-monsoon season
	(*d*) In basin irrigation	(*D*) water is applied to the surface of the soil in the form of a spray.
(*ii*)	(*a*) In an aqueduct	(*A*) the drainage water is carried in a trough supported on piers while their irrigation canal passes below with its FSL below the slab of the trough above.
	(*b*) In a level crossing	(*B*) the drainage water is carried in a trough supported on piers while the irrigation canal passes below with its FSL above the slab of the trough above
	(*c*) In a super-passage	(*C*) the canal and the drain approach each other practically at the same level
	(*d*) In a syphon	(*D*) the irrigation canal is carried across the drain in a trough supported on piers with the canal bed level well above the HFL of the drain

(*iii*)	(*a*) Surface run-off is that form of	(*A*) precipitation in which the drops of water are under 0.5 mm in diameter and intensity is usually less than 1 mm per hour
	(*b*) Infiltration is that form of	(*B*) the hydrologic cycle in which water moves from the surface of the ground into the soil
	(*c*) Drizzle is that form of	(*C*) precipitation in which the water drops are greater than 0.5 mm diameter
	(*d*) Rain is that form of	(*D*) precipitation which travels over land surface or just below it
(*iv*)	(*a*) For an earthen dam	(*A*) the rock in the abutment must be stronger than in the foundation
	(*b*) For an arch dam	(*B*) the foundation may be rock or soil
	(*c*) For a solid gravity dam	(*C*) the foundation should consist of pervious material
	(*d*) For a barrage	(*D*) the foundation rock should be stronger than in the abutments
(*v*)	(*a*) Rabi season	(*A*) last from Ist April to 30th September
	(*b*) Kharif season	(*B*) is the total quantity of water needed for growth of crop to maturity
	(*c*) Delta	(*C*) is the relation between the volume of water and the area of the crop which it matures
	(*d*) Duty	(*D*) starts on Ist October and ends on 31st March
(*vi*)	(*a*) Undersluices	(*A*) regulates the supply of water in the canal
	(*b*) Canal head regulator	(*B*) enable the fish to migrate to their spawning grounds upstream of the barrage
	(*c*) Barrage bays	(*C*) preserve a clear and defined river channel approaching the canal regulator
	(*d*) Fish ladder	(*D*) are operated to release the excess water during floods
(*vii*)	(*a*) Cross drainage works include	(*A*) notches, escapes, sluices and syphon well drops
	(*b*) Works for regulation of water levels include	(*B*) bridges and culverts
	(*c*) Communication works include	(*C*) spurs, groynes, bunds, mattersses and aprons
	(*d*) River training works include	(*D*) aqueducts, syphons, syphon, aqueducts, superpassages and level crossings

Answers to Questions of 1993 (*Winter*) Exam.

A (*i*) – Lift **A** (*ii*) – saline **A** (*iii*) – rigid module or modular outlet

A (*iv*) – more **A** (*v*) – spillway **A** (*vi*) – transpiration loss

B (*i*)– (*d*) **B** (*ii*) – (*c*) **B** (*iii*) – (*c*)

B (*iv*) – (*d*) **B** (*v*) – (*a*) **B** (*vi*) –(*c*)

B (*vii*)– (*a*)

C (i) : (*a*)–(*C*), (*b*)–(*A*)
(*c*)–(*D*), (*d*)–(*B*)

C (ii) : (*a*)–(*D*), (*b*)–(*C*)
(*c*)–(*A*), (*d*)–(*B*)

C (iii) : (*a*)–(*D*)
(*b*)–(*B*)
(*c*)–(*A*)
(*d*)–(*C*)

C (iv) : (*a*)–(*B*)
(*b*)–(*A*)
(*c*)–(*D*)
(*d*)–(*C*)

C (v) : (*a*)–(*D*)
(*b*)–(*A*)
(*c*)–(*B*)
(*d*)–(*C*)

C (vi) : (*a*)–(*C*)
(*b*)–(*A*)
(*c*)–(*D*)
(*d*)–(*B*)

Year 1994 (*Summer*) Exam.

Q. Choose the correct answer for the following :

(*i*) Evaporation is measured by using

(*a*) an open pan (*b*) lysimeter

(*c*) infiltrometer (*d*) a neutron tube

(*ii*) The Standard Symon's Type Rain Gauge has a collecting a diameter

(*a*) 10 cm (*b*) 12.7 cm

(*c*) 5.08 cm (*d*) 25.4 cm

(*iii*) The average annual rainfall over whole of India is estimated as :

(*a*) 189 cm (*b*) 319 cm

(*c*) 89 cm (*d*) 117 cm

(*iv*) An Isohyet is a line joining points having

(*a*) equal evaporation value

(*b*) equal barometric pressure

(*c*) equal height above MSL

(*d*) equal rainfall depth in a given duration

(*v*) A coffer dam is

(*a*) kind of gravity dam

(*b*) an earthen dam of small height built to protect important areas

(*c*) a temporary structure constructed to exclude water from the work area during construction.

(*d*) an embankment built along a river to regulate the river

(*vi*) In an Inundation Irrigation System there will
- (*a*) be neither a diversion work nor a regulator
- (*b*) be only a diversion structure across the river
- (*c*) be both a regulator on the canal as well as a diversion structure across the river

(*vii*) In well drained soil, the useful soil moisture for plant growth essentially comes from
- (*a*) gravity water
- (*b*) capillary water
- (*c*) hygroscopic water
- (*d*) water of adhestion

(*viii*) A land is known as waterlogged when
- (*a*) the *p*H of soil reaches a value of 8.5
- (*b*) the soil in the root zone has high salinity
- (*c*) the soil pores in the root are saturated with water
- (*d*) there is water flowing on the surface of the land

(*ix*) Irrigation water having an SAR value of 20 is called as
- (*a*) very high sodium water
- (*b*) high sodium water
- (*c*) medium sodium water
- (*d*) low sodium water

(*x*) Alkali soils are reclaimed by
- (*a*) leaching only
- (*b*) addition of gypsum and leaching
- (*c*) addition of gypum only
- (*d*) provision of drainage

(*xi*) A groyne pointing upstream in a river is known as
- (*a*) attracting groyne
- (*b*) repelling groyne
- (*c*) fending groyne
- (*d*) Denehy's grone

(*xii*) The chemical suitable for use as a water evaporation retarding and reducing agent is
- (*a*) butyl alcohol
- (*b*) methyl alcohol
- (*c*) cetyl alcohol
- (*d*) ethyl alcohol

(*xiii*) The chemical dilution method of stream gauging is ideally suited for measuring discharges in
- (*a*) a large alluvial river
- (*b*) steady flow in a small turbulent stream
- (*c*) a stretch of a river having heavy pollution
- (*d*) a tidal estuary

(*xiv*) Consumptive use is
- (*a*) the water used up in plant metabolism
- (*b*) the sum of evapotranspiration and amount used up in plant metabolism
- (*c*) the sum of evapotranspiration and infiltration losses
- (*d*) the combined use of surface and ground water resources

(*xv*) Conjunctive use of water in a basin means
- (*a*) combined use of water for irrigation and for hydro-power generation
- (*b*) combined use of surface and groundwater resources
- (*c*) use of irrigation water for both Rabi and Kharif crops
- (*d*) use of irrigation water by a cooperative of farmers

(*xvi*)* Permanent wilting point is
- (*a*) a characteristic of the plant
- (*b*) a soil characteristic
- (*c*) a soil characteristic modified by the crop
- (*d*) dependent on soil water plant fertiliser interaction

(*xvii*) A hytograph is a plot of
- (*a*) cumulative rainfall versus time
- (*b*) rainfall depth versus duration
- (*c*) evaporation rate versus time
- (*d*) rainfall intensity versus time

(*xviii*) The doubles mass analysis is adopted to
- (*a*) estimate the missing rainfall data
- (*b*) obtain intensities of rainfall at various duration
- (*c*) check the consistency of data
- (*d*) obtain the amount of storage needed to maintain a demand pattern

(*xix*) There is minimum wastage of water in
- (*a*) furrow irrigation
- (*b*) check basin irrigation
- (*c*) sprinkler irrigation
- (*d*) border method of irrigation

(*xx*) The coefficient of roughness for an earthen canal in excellent condition is
- (*a*) 0.01
- (*b*) 0.0225
- (*c*) 0.025
- (*d*) 0.7.

Answers to Questions of 1994 (*Summer*) Exam

(*i*)–(*a*)	(*ii*)–(*b*)	(*iii*)–(*d*)
(*iv*)–(*c*)	(*v*)–(*c*)	(*vi*)–(*c*)
(*vii*)–(*b*)	(*viii*)–(*c*)	(*ix*)–(*b*)
(*x*)–(*b*)	(*xi*)–(*b*)	(*xii*)–(*c*)
(*xii*)–(*b*)	(*xiv*)–(*b*)	(*xv*)–(*b*)
(*xvi*)–(*b*)	(*xvii*)–(*d*)	(*xviii*)–(*c*)
(*xix*)–(*c*)	(*xx*)–(*b*)	

Hints for Solving Starred Questions of Year 1994 (*Summer*) Exam

Q. (*xvi*) *Permanent wilting percentage or permanent wilting point or wilting coefficient is a soil characteristic, since all plants whose root systems thoroughly permeats the soil wilt at nearly the same soil moisture content ; when grown in a particular soil in a humid atmosphere.*

Year 1994 (*Winter*) Exam.

Q. (*A*) Fill in the blanks :

(*i*) A 6-hours storm had 6 cm of rainfall and the resulting runoff was 3 cm. If the ϕ-index remains at the same value, the runoff due to 12 cm of rainfall in 9 hours in the catchment is .... cm.

(*ii*) The duty for a crop with a base period of 140 days is 3456 hectare per cumec. The delta for the crop will be ..... m.

(*iii*) A one-day rainfall of 10.0 cm at a place was found to have return period of 100 years. The probability that a one day rainfall of this or larger magnitude will occur in the next year is .....

(*B*) Choose the correct answer for the following :

(*i*) Retrogression of downstream levels, generally considered in the design of weirs or barrages, is

(*a*) higher at high flood stage than at low water levels
(*b*) same at high flood stage and at low water levels
(*c*) higher at low water levels stage than at high flood stage
(*d*) independent of the stage of flow.

(*ii*) The average pan coefficient for the standard US Weather Bureau class A pan is

(*a*) 0.55 (*b*) 0.70
(*c*) 0.85 (*d*) 0.95.

(*iii*) To determine discharge of a stream from rating curve, at a section, one needs

(*a*) current meter readings at various verticals at the section
(*b*) slope of the water surface at the section
(*c*) stage at the section
(*d*) surface velocity at various sections.

(*iv*) The useful moisture of a soil is equal to its

(*a*) field capacity
(*b*) saturation capacity
(*c*) moisture content at permanent wilting point
(*d*) difference between field capacity and permanent wilting point within the root zone of plants.

(*v*) Distribution outlet is said to be proportional, if its flexibility is

(*a*) equal to unity (*b*) less than unity
(*c*) slightly greater than unity (*d*) equal to two.

(*vi*) If average particle size of silt in millimetres is m, then Lacey's silt factor (f) is proportional to

(*a*) m^2 (*b*) $m\, m^{1/2}$
(*c*) $m^{\frac{1}{2}}$ (*d*) $m^{1/3}$.

(*vii*) The basic assumptions of unit hydrograph theory are
- (*a*) linear response and non-linear time variance
- (*b*) linear response and time invariance
- (*c*) non-linear response and time invariance
- (*d*) linear response and linear time variance.

(*viii*) The monthly rainfall at a place is generally indicated as above or below *normal*. The term *normal* means
- (*a*) the rainfall in the same month of the previous year
- (*b*) the rainfall expected based on previous month's data
- (*c*) the average rainfall of the previous twelve months
- (*d*) the average monthly rainfall for the same month computed from 30 years of past record.

(*ix*) The optimum depth of kor-watering for rice crop, is
- (*a*) 22 cm (*b*) 19 cm
- (*c*) 16 cm (*d*) 13 cm.

(*x*) The main function of division headworks provided at the off-take of a canal from a river, is
- (*a*) to raise the water level in the river
- (*b*) to control the floods
- (*c*) to store water
- (*d*) to control entry of silt into the canal.

(*xi*)* The downstream expansion head of a guide bank is extended to subtend an angle at the centre between
- (*a*) 30° and 45° (*b*) 45° and 60°
- (*c*) 60°and 75° (*d*) 75° and 90°.

(*xii*)* Direct runoff is made up of
- (*a*) surface runoff, prompt interflow and channel precipitation
- (*b*) surface runoff, infiltration and evapotranspiration
- (*c*) overland flow and quick return flow
- (*d*) infiltration, rainfall and evaporation.

(*xiii*)* The velocity of drainage water in the barrels of syphon aqueduct is normally limited to
- (*a*) 1 to 2 m per second (*b*) 2 to 3 m per second
- (*c*) 3 to 4 m per second (*d*) 4 to 5 m per second.

(*xiv*) Cross-regulators in main canals are provided to
- (*a*) regulate water supply in the off-taking channel
- (*b*) regulate water supply in the main channel
- (*c*) regulate excessive flood water
- (*d*) head up water for adequate supply into the off-taking channel.

Answers to Question of 1994 (*Winter*) Exam.

A (*i*) – 7.5	*A* (*ii*) – 0.35	*A* (*iii*) – 0.01
B (*i*) – (*c*)	*B* (*ii*) – (*b*)	*B* (*iii*) – (*c*)
B (*iv*) – (*d*)	*B* (*v*) – (*a*)	*B* (*vi*) – (*c*)
B (*vii*) – (*b*)	*B* (*viii*)– (*d*)	*B* (*ix*) – (*b*)
B (*x*) – (*c*)	*B* (*xi*)– (*b*)	*B* (*xii*)– (*c*)
B (*xiii*) – (*b*)	*B* (*xiv*)– (*d*)	

Hints for Solving Starred Questions of 1994 (*Winter*) Exam

(*xi*) Please refer Fig. 8.19 on page 583

(*xii*) D.R.O = S.R.O = True S.R.O. + Sub surface storm flow or quick return flow

(*xiii*) Please refer articles 14.5.2 and 14.5.3 on page 850

Year 1995 (*Winter*) Exam

Q. (*A*) Choose the correct answer for the following :

(*i*) Conjuctive use of water in a basin means ;

(*a*) Combined use of water for irrigation and for hydropower generation ;

(*b*) Combined use of surface and ground water resources ;

(*c*) Use of irrigation water for both rabi and kharif season ;

(*d*) Use of irrigation water by cooperative of farmers.

(*ii*) Consumptive use is

(*a*) the water used up in plant metabolism

(*b*) the sum of evapotranspiration and the amount used up in plant metabolism;

(*c*) the sum of evapotranspiration and infiltration losses ;

(*d*) the combined use of surface and ground water resources.

(*iii*) A plot between rainfall intensity versus time is called

(*a*) mass curve ; (*b*) hyetograph ;

(*c*) isohyet ; (*d*) hydrograph.

(*iv*) The volume of water that can be extracted by force of gravity from a unit volume of aquifer material is known as

(*a*) specific yield ; (*b*) safe yield ;

(*c*) specific retention (*d*) specific storage.

(*v*) For plant's growth useful soil moisture is

(*a*) capillary water ; (*b*) gravity water ;

(*c*) hygroscopic water ; (*d*) chemical water.

(*vi*) The normal average annual rainfall over the whose of India is estimated as

(*a*) 88 cm ; (*b*) 217 cm ;

(*c*) 119 cm ; (*d*) 290 cm.

(*vii*)* The minimum free board recommended for lined canals carrying a discharge greater than 10 m^3/s is

(*a*) 0.15 m ; (*b*) 0.60 m ;

(*c*) 1.25 m ; (*d*) 0.75 m.

(*viii*) A land is known as water logged when

(*a*) the pH of the soil reaches a value of 8.5 ;

(*b*) the soil in the root zone has high salinity

(*c*) the soil pores in the root zone are saturated with water ;

(*d*) there is water flowing on the surface of the land.

(*ix*) A characteristic feature of a barrage is

(*a*) the provision of a raised crest ;

(*b*) the creation of a storage resevoir on the upstream side ;

(*c*) the provision of a series of gates across the river for flow regulation ;

(*d*) that it is built in delta areas only.

(*x*) The use of rational formula for estimating floods is limited to catchments of sizes less than

(*a*) 5000 km^2 ; (*b*) 500 km^2 ;

(*c*) 50 km^2 ; (*d*) 5 km^2.

(*xi*) The average pan coefficient for the standard U.S. Weather Bureau class *A* pan is

(*a*) 0.60 ; (*b*) 0.65 ;

(*c*) 0.70 ; (*d*) 0.75.

(*xii*) The standard Symon's type Rain Gauge has a collecting area of diameter

(*a*) 5.08 cm ; (*b*) 10 cm ;

(*c*) 12.7 cm ; (*d*) 25.4 cm.

(*xiii*) A medium head Hydroelectric Scheme is one where

(*a*) Gross head is between 15 m to 60 m ;

(*b*) Net head operating on turbine is between 15 m to 60 m ;

(*c*) Total loss of head in the penstocks is between 15 m to 60 m.

(*d*) The difference in the level of tail water and centre line of turbine is between 15 m to 60 m.

(*xiv*) Dicken's formula is the form of

(*a*) $Q = CM^{1/3}$; (*b*) $Q = CM^{2/3}$;

(*c*) $Q = CM^{1/4}$; (*d*) $Q = CM^{3/4}$.

where M is the catchment Area

(*B*) Fill in the blanks :

(*i*)* A 2 hour unit hydrograph of a catchment is triangular in shape with a time base of 60 hour and peak ordinate of 30 m^3/s. The area of the catchment is ..... km^2

(*ii*) The delta for a crop with a base period of 150 days is 50 cm. The duty for the crop will be ..... hectares per cumec.

*(iii)** A mean annual runoff of 1 m^3/s from a catchment of area 10 km^2 represents an effective rainfall ..... cm.

Answers to Questions o 1995 (*Winter*) Exam

A (i) – (*b*)	*A (ii)*– (*b*)	*A (iii)*– (*b*)
A (iv) – (*a*)	*A (v)*– (*a*)	*A (vi)*– (*c*)
A (vii) – (*d*)	*A (viii)*– (*c*)	*A (ix)*– (*c*)
A (x) – (*c*)	*A (xi)*– (*c*)	*A (xii)*– (*c*)
A (xiii)– (*b*)	*A (xiv)*– (*d*)	*B (i)* – 648
B (ii)– 2592	*B (iii)*– 315.36	

Hints for Solving Starred Questions of 1995 (*Winter*) Exam

Q. A (*vii*) Please refer Table 4.11

Q. B (*i*) $$\frac{\frac{30\ m^3}{s} \times (60 \times 60 \times 60s) \times 100}{\text{Catchment area } A \text{ in m}^2} = 1 \text{ cm}$$

$$\therefore A = 648 \times 10^6 \text{ m}^2 = 648 \text{ km}^2$$

Q. B (*iii*) Vol. of water in 1 yr $= \frac{1\ m^3}{s} \times (365 \times 24 \times 60 \times 60$ secs in a yr

$$= 315.36 \times 10^5 \text{ m}^3$$

$$\text{Eff. rainfall depth} = \frac{\text{Volume of water}}{\text{Area}} = \frac{315.36 \times 10^5\ m^3}{10 \times 10^6 \text{ m}^2}$$

$$= \frac{315.36}{10^2} \text{m} = 315.36 \text{ cm.} \quad \textbf{Ans.}$$

Year 1996 (*Summer*) Exam

Q. (*A*) Fill in the blanks :

*(i)** A canal is 80 km long and has an average surface width of 15 m. If the average evaporation measured in a standard U.S. Weather Bureau Class, a pan is 0.5 cm/day, the volume of water evaporation in the month of September is ..... cubic metres.

*(ii)** If the wind velocity at a height of 2 m above ground is 5.0 km per hour, its value at a height of 10 m above ground can be expected to be about ..... km per hour.

*(iii)** In a triangular channel the top width and depth of flow were 2.0 m and 0.9 m respectively. Velocity measurements on the centre line at 18 cm and 72 cm below the free water surface indicated velocities of 0.6 m/s and 0.4 m/s respectively. The discharge in the channel is ..... cumec.

*(iv)** During a flood in a wide rectangular channel it is found that at a section the depth of flow increases by 50 per cent and at this depth the water surface slope is half its original value in a given interval of time. This marks an approximate change in the discharges of ..... per cent.

(*v*)* A triangular direct runoff hydrograph due to a storm has a time base of 80 hours and a peak flow of 50 m^3/s occurring at 20 hours from the start. If the catchment area is 144 km^2, the rainfall excess in the strom was ..... cm.

(*B*) Choose the correct answer for the following :

(*i*) Cold front precipitation is :

(*a*) very intense and of short duration
(*b*) very intense and of considerable duration
(*c*) light and of short duration
(*d*) Showery and of considerable duration.

(*ii*) A hyetograph is a plot against time of :

(*a*) cumulative rainfall (*b*) Rainfall intensity
(*c*) discharge (*d*) stage.

(*iii*) The mass curve of rainfall of a storm is plot of :

(*a*) rainfall depths for various equal durations plotted in decreasing order
(*b*) rainfall depths for various equal durations plotted in increasing order
(*c*) rainfall intensity *vs.* time in chronological order
(*d*) accumulated precipitation *vs.* time in chronological order.

(*iv*) The slope-area method is extensively used in :

(*a*) development of rating curve
(*b*) estimation of flood discharge based on high-water marks
(*c*) cases where shifting control exist
(*d*) cases where backwater effect is present.

(*v*) The term base flow denotes delayed flow reaching a stream from :

(*a*) groundwater only
(*b*) ground water and snow melt
(*c*) groundwater and inter flow
(*d*) groundwater, snow melt and inteflow.

(*vi*) A module is said to be 'proportional' when 'flexibility' equals :

(*a*) 0.8 (*b*) 0.9
(*c*) 1.0 (*d*) 1.1.

(*vii*)* For consolidation grouting, depth of *B*-holes drilled through the foundation rock generally varies between :

(*a*) 5 and 10 m (*b*) 10 and 15 m
(*c*) 15 and 20 m (*c*) 20 and 25 m.

(*viii*)* According to Gales the ratio of upstream length of guide bank to the length of structure between the aboutments for flood discharges more than 20000 cumecs shall be :

(*a*) 1.3 (*b*) 1.4
(*c*) 1.5 (*d*) 1.6.

(*ix*) For row crops, e.g., potatoes, sugarbeets, etc. the most commonly adopted method of surface irrigation is :

(*a*) check flooding (*b*) free flooding
(*c*) basin flooding (*d*) furrow and corrugation irrigation.

(*x*) According to Khosla, to keep the structure safe against piping, exit gradient to be provided lies between :

(*a*) 0.10 and 0.15 (*b*) 0.15 and 0.20

(*c*) 0.20 and 0.26 (*d*) 0.25 and 0.30.

Answers to Questions of 1996 (*Summer*) Exam

A (*i*)– 126000	*A* (*ii*)– 6.29	*A* (*iii*)– 0.45
A (*iv*)– 39	*A* (*v*)– 5	
B (*i*)– (*a*)	*B* (*ii*)– (*b*)	*B* (*iii*)– (*d*)
B (*iv*)– (*b*)	*B* (*v*)– (*a*)	*B* (*vi*)– (*c*)
B (*vii*)– (*b*)	*B* (*viii*)– (*c*)	*B* (*ix*)– (*d*)
B (*x*)– (*c*)		

Hints for Solving Starred Questions of 1996 (*Summer*) Exam

Q. A (*i*) Actual Evaporation from canal during September month

$$= (\text{Area of Canal}) \times \text{Evaporation rate} \times \text{No of days} \times \text{Pan coeff.}$$

$$= (80\text{ km} \times 15\text{ m})\left(\frac{0.5\text{ m}}{100\text{ day}} \times 0.7\right) \times 30\text{ days}$$

$$= \left[80000 \times 15 \times \frac{0.35}{100} \times 30\text{ m}^3\right] = \mathbf{126000\ m^3}\ \textbf{Ans.}$$

Q. A (*ii*) Using Eqn. (7.43 a), we have

$$\frac{V_{z1}}{V_{z2}} = \left(\frac{z_1}{z_2}\right)^{0.143}$$

where $V_{z1} = V_{2m} = 5.0$ km/hr

$z_1 = 2$ m

$z_2 = 10$ m

$$\therefore \frac{V_{2m}}{V_{10m}} = \left(\frac{2}{10}\right)^{0.143} = \frac{5.0\text{ km/h}}{V_{10}}m$$

or $$V_{10m} = \frac{5\text{ km/h}}{0.794} = \mathbf{6.29\ km/h}\ \textbf{Ans.}$$

Q. A (*iii*) Vel. at 0.2y depth =- vel at 0.18 m depth = 0.6 m/s

Vel. at 0.8y depth = vel. at 0.72m depth = 0.4 m/s

$$\text{Av. mean velocity} = \frac{0.6 + 0.4}{2} = 0.5\text{ m/s}$$

$$\text{Area of channel}(A) = \frac{1}{2} \times (2.0\text{ m} \times 0.9\text{ m}) = 0.9\text{ m}^2$$

Discharge $Q = V.A = 0.5 \text{ m/s} \times 0.9 \text{ m}^2$

$= 0.45 \text{ m}^3/\text{s} = \mathbf{0.45 \text{ cumecs}}$ **Ans.**

Q. A (*iv*) For wide rectangular channel.

$$Q = \frac{1}{n} \cdot A.R^{2/3} \sqrt{S_f}$$

$$= \frac{1}{n} \cdot (B \cdot y) \cdot (y)^{2/3} \cdot \sqrt{S_f}$$

or $$Q = \frac{1}{n} \cdot y^{5/3} \cdot \sqrt{S_f}$$

$\therefore$ $$\frac{Q_2}{Q_1} = \frac{y_2^{5/3} \cdot \sqrt{S_{f_2}}}{y_1^{5/3} \cdot \sqrt{S_{f_1}}}$$

But $y_2 = 1.5\, y_1$

$$S_{f_2} = \frac{1}{2} S_{f_1} = 0.5\, S_{f_1}$$

$\therefore$ $$\frac{Q_2}{Q_1} = \frac{(1.5)^{5/3} \cdot y_1^{5/3} \cdot \sqrt{0.5} \cdot \sqrt{S_{f_1}}}{y_1^{5/3} \cdot \sqrt{S_{f_1}}} = (1.5)^{5/3} \cdot \sqrt{0.5} = 1.39$$

%age change in discharge $= \frac{Q_2 - Q_1}{Q_1} \times 100$

$$= \left(\frac{Q_2}{Q_1} - 1\right) 100 = (1.39 - 1)\, 100 = \mathbf{39\%}$$ **Ans.**

Q. A (*v*) Vol. of water contained in the U.H =

$$= \frac{1}{2} \times 50 \text{ m}^3/\text{s} \times 80 \text{ h}$$

$$= \frac{2000\, m^3 \cdot h}{s} = 2000\, m^3 \times 3600 \frac{s}{s}$$

$$= 7.2 \times 10^6\, m^3.$$

$$\text{Depth of water} = \frac{\text{Volume of water}}{\text{C.A}} = \frac{7.2 \times 10^6\, m^3}{144 \times 10^6\, m^3}$$

$= 0.05 \text{ m} = 5 \text{ cm}$ **Ans.**

Q. B (*vii*) Please refer article 19.15 (2).

Q. B. (*viii*) Please refer article 8.5.3 (2) (*iii*).

Year 1996 (*Winter*) Exam

Q. (*A*) Choose the correct answer in the following

(*i*) The interval of supply of irrigation water depends on the type of

(*a*) crop only (*b*) soil and crop

(*c*) crop and climate (*d*) soil, crop and climate.

(*ii*) Increase in the duration of the unit hydrograph causes an increase in the
(*a*) peak discharge (*d*) rainfall intensity
(*c*) equilibrium discharge (*d*) time base.

(*iii*) Fish ladder is provided on the side of
(*a*) divide wall (*b*) diaphragm wall
(*c*) core wall (*d*) wing wall.

(*iv*) The volume of water stored between normal reservoir level and the maximum reservoir level is known as
(*a*) live storage (*b*) surcharge storage
(*c*) valley storage (*d*) bank storage.

(*v*) Consumptive use is the sum of
(*a*) evapotranspiration and infiltration losses
(*b*) evapotranspiration plus application losses
(*c*) evapotranspiration plus the amount of water used up in plant metabolism
(*d*) evapotranspiration plus the amount of water used up in plant metabolism plus percolation losses.

(*vi*) Other factors remaining same, the open water evaporation rate increases as
(*a*) sunshine hours increase
(*b*) altitude decreases
(*c*) barometric pressure increases
(*d*) salt content of water decreases.

(*vii*) For a barrage, the exit gradient is independent of
(*a*) the applied head of water
(*b*) the horizontal length of floor
(*c*) the depth of upstream cut-off
(*d*) the depth of downstream cut-off.

(*viii*) A syphon aqueduct is provided when
(*a*) the canal bed level is well above the HFL of the drain
(*b*) the canal bed level lies between the bed level and HFL of the drain
(*c*) the drain bed level is well above the FSL of the canal
(*d*) the drain bed level lies between the bed level and FSL of the canal.

(*ix*) An inundation irrigation system requires the construction of
(*a*) only a diversion structure
(*b*) only a canal regulator
(*c*) neither a diversion structure nor a canal regulator
(*d*) both a diversion structure and a canal regulator.

(*x*) In order to reduce uplift on a gravity dam, the type of grouting done is
(*a*) curtain grouting near the heel
(*b*) consolidation grouting near the heel
(*c*) curtain grouting near the toe
(*d*) consolidation grouting near the toe.

(*xi*) In order to keep the saturation line in a canal embankment well within the toe, it is necessary to provide

(*a*) spoil bank (*b*) counterberm

(*c*) free board (*d*) berm.

(*xii*) The term piping used in connection with diversion structures on permeable soil is associated with

(*a*) drainage of seepage water

(*b*) measurement of uplift pressure

(*c*) failure initiated by boiling

(*d*) consolidation of foundation.

(*xiii*) The upstream face of an earth dam is

(*a*) an equipotential line (*b*) a streamline

(*c*) a phreatic line (*d*) a streak line

(*xiv*) Silt ejector is provided on

(*a*) the river, just upstream of the undersluices

(*b*) the canal, just downstream of the canal head regulator

(*c*) the river, some distance downstream of the undersluices

(*d*) the canal, some distance downstream of the canal head regulator

(*B*) Fill in the blanks :

(*i*)* The area of a drainage basin is twice that on an adjoining one. Excepting area, other characteristics of the two basins are identical. Q1 and Q2 are the peak discharges, computed by Dicken's formula, for the smaller and larger basin respectively. Then $Q_2 = Q_1 \times$ .....

(*ii*)* In a field measuring 30 hectares, 40 cm of water was stored in the root zone when 6 cumecs of water was supplied for 8 hours. The field application efficiency is .....

(*iii*)* he dominant discharge of a river in alluvial plain is 3600 cumecs. The water-way on this river, according to Lacey's formula, should be ..... m.

Answers to Questions of 1996 (*Winter*) Exam.

A (*i*) – (*d*)	*A* (*ii*) – (*d*)	*A* (*iii*) – (*a*)
A (*iv*) – (*b*)	*A* (*v*) – (*c*)	*A* (*vi*) – (*d*)
A (*vii*)– (*c*)	*A* (*viii*)– (*b*)	*A* (*ix*) – (*d*)
A (*x*) – (*a*)	*A* (*xi*) – (*b*)	*A* (*xii*)– (*c*)
A (*xiii*)– (*a*)	*A* (*xiv*)– (*d*)	
B (*i*) – 1.6818	*B* (*ii*) – 69.44%	*B* (*iii*)– 285

Hints for Solving Questions of 1996 (*Winter*) Exam.

Q. A (*ix*) Please refer article 19.15 (2) (*b*) at page 1167

Q. B (*i*) $Q_1 = C \cdot A_1^{3/4}$

where Q_1 is the discharge of smaller basin

$$Q_2 = C \cdot A_2^{3/4} = C \cdot (2A_1)^{3/4}$$

$$\frac{Q_2}{Q_1} = (2)^{3/4} = 1.6818$$

or $Q_2 = Q_1 \times 1.6818$ **Ans.**

Q. B. (*ii*) Vol. of water actually stored in root zone

$$= (30 \times 10^4 \text{ m}^2)\ (0.4\ m) = 12 \times 10^4 \text{ m}^3$$

Vol. of water actually supplied

$$= 6 \text{ m}^3/s\ (8 \times 60 \times 60s)$$

$$= 48 \times 3600 = 172800 \text{ m}^3 = 17.28 \times 10^4 \text{ m}^3$$

$$\text{Field application efficiency} = \frac{\text{Vol. of water actually stored}}{\text{Vol. of water supplied}}$$

$$= \frac{12 \times 10^4}{17.28 \times 10^4} = 0.6944 = \mathbf{69.44\%} \quad \textbf{Ans.}$$

Q. B. (*iii*) From Eqn. (15.1),

$$L = 4.75\sqrt{Q} = 4.75 \times \sqrt{3600} = 4.75 \times 60 = \mathbf{285\ m.} \quad \textbf{Ans.}$$

Year 1997 (*Summer*) Exam

(*i*) An arch dam is constructed with

(*a*) Earth (*b*) Concrete
(*c*) Boulders (*d*) None of the above

(*ii*) A core wall is provided in

(*a*) An earth dam (*b*) A canal regulator
(*c*) An aqueduct (*d*) A gravity dam

(*iii*) A hydrograph is a relation between time in hours and

(*a*) Rainfall in mm (*b*) Percolation in cm
(*c*) Surface runoff in cumec (*d*) Evaporation in cm

(*iv*) The self weight of a gravity dam

(*a*) Favours stability (*b*) Causes overturning
(*c*) Causes sliding (*d*) Causes uplift

(*v*) A level crossing is a cross-drainage work where

(*a*) Canal flows under a specified level above the drain
(*b*) Canal flows under pressure
(*c*) Canal has a free flow
(*d*) None of the above

(*vi*) Evaporation is maximum

(*a*) When the temperature is very low
(*b*) When it is raining
(*c*) When the temperature is high
(*d*) During night

(*vii*) Other conditions remaining the same, runoff is maximum from precipitation occurring over

(*a*) Sandy strata

(*b*) Rocky area

(*c*) Alluvial tracts after a dry spell

(*d*) Boulders

(*viii*) Waterlogging is eliminated by

(*a*) Deep ploughing (*b*) Shallow ploughing

(*c*) Irrigation (*d*) Providing tile drains

(*ix*) Lacey's theory is applicable to flow

(*a*) Through pipes

(*b*) Over spillways

(*c*) In alluvial rivers and canals

(*d*) In concrete lined channels

(*x*) Lift irrigation is flow

(*a*) By gravity (*b*) From lower level to higher level

(*c*) By percolation (*d*) Through sprinkler heads

(*xi*) Critical velocity ratio is used in connection with the design of

(*a*) Alluvial canals (*b*) Earth dams

(*c*) Concrete lined canals (*d*) Barrages

(*xii*) Base flow separation is used in connection with

(*a*) Seepage flow (*b*) Infiltration

(*c*) Evaporation (*d*) Stream flow

(*xiii*) The time base of a hydrograph increases with

(*a*) Increase in intensity of the storm

(*b*) Decrease in duration of the storm

(*c*) Increase in time of concentration

(*d*) Decrease in infiltration capacity of the storm

(*xiv*) Thiessen polygon is used in connection with

(*a*) Average rainfall determination

(*b*) Catchment area computation

(*c*) Drawing hydrographs

(*d*) Estimating infiltration

(*B*) Fill in the blanks in the following statements :

(*i*) The depth of scour for a discharge intensity of 5 cumec/m and a silt factor of 1 is ____ m.

(*ii*) Friction loss through a pipe 100 m long, 1.5 m diameter, carrying a discharge of 6 cumec and rugosity coefficient of 0.014 is ____ m.

Answers to Questions of 1997 (*Summer*) Exam

A (*i*)–(*b*)	*A* (*ii*)–(*a*)	*A* (*iii*)–(*c*)	*A* (*iv*)–(*a*)
A (*v*)–(*d*)	*A* (*vi*)–(*c*)	*A* (*vii*)–(*b*)	*A* (*viii*)–(*d*)
A (*ix*)–(*c*)	*A* (*x*)–(*b*)	*A* (*xi*)–(*a*)	*A* (*xii*)–(*d*)
A (*xiii*)–(*c*)	*A* (*xiv*)–(*a*)	*B* (*i*)–3.95	*B* (*ii*)–1.52

Hints for Solving Starred Questions 1997 (*Summer*) Exam

Q. B (*i*) $R = \text{Scour depth} = 1.35\left(\frac{q^2}{f}\right)^{1/3} = 1.35\left(\frac{5^2}{1}\right)^{1/3} = \mathbf{3.95\ m}$ **Ans.**

Q. B (*ii*) From Manning's formula, we have

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{S}$$

or $$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot \sqrt{\frac{H_L}{L}}$$

or $$\frac{n^2 Q^2 L}{A^2 \cdot R^{4/3}} = H_L$$

or $$H_L = \frac{n^2 \cdot Q^2 \cdot L}{\left(\frac{\pi}{4} d^2\right) \cdot \left(\frac{d}{4}\right)^{4/3}} = \frac{(0.014)^2 \cdot (6)^2 \times 100}{\left(\frac{\pi}{4} \times 1.5^2\right)^2 \left(\frac{1.5}{4}\right)^{4/3}} = \mathbf{1.52\ m}$$ **Ans.**

Year 1997 (*Winter*) Exam

Q. A. *Fill in the blanks*

(*i*) Field capacity is defined as _____.

(*ii*) The delta for the crop with duty 864 hectares/cumec on the field and base period of 120 days is _____ cm.

(*iii*) Infiltration capacity is more than infiltration rate when _____.

(*iv*) A hyetograph is the plot of _____.

(*v*) In Kennedy's equation for non-silting non-scouring velocity, the critical velocity ratio was introduced to take into consideration the effect of _____.

(*vi*) Syphon aqueduct is the cross-drainage work provided to carry canal over natural drain when _____.

(*vii*) The ϕ_{index} is defined as the _____.

Q. B. *Choose the correct statement for the following :*

(*i*) Ravi crops are sown and harvested in the month of :

(*a*) April and September (*b*) October to March

(*c*) February and June (*d*) January and May

(*ii*) Mixed cropping is defined as :

(*a*) two or more crops grown during any year
(*b*) two or more crops grown during the same crop season in different fields
(*c*) two or more crop seasons of the year
(*d*) growing of two ore more crops together in the same field during the same crop season

(*iii*) For the upstream slope of an earth dam, the most critical condition is :

(*a*) sudden drawdown condition
(*b*) steady seepage condition
(*c*) neither sudden drawdown nor steady seepage condition
(*d*) during construction when the reservoir is allowed to be filled

(*iv*) Seepage endangers the stability of an earth dam built on pervious foundation because of piping which depends on :

(*a*) height of the dam (*b*) quantity of seepage flow
(*c*) value of exist gradient (*d*) total storage capacity of reservoir

(*v*)* If a river in alluvial plain has a dominant discharge of 1600 cumecs, the waterway for a bridge on the river should be approximately

(*a*) 250 m (*b*) 240 m
(*c*) 190 m (*d*) 180 m

(*vi*) For no tension at any point on a gravity dam, the criterion to be satisfied is :

(*a*) the resultant of all the forces must always pass through the mid point of the base of the dam
(*b*) the resultant force for all conditions of loading must pass through the middle third of the base
(*c*) the resultant of all forces must pass through the upstream extremity of the middle third of the base
(*d*) the resultant of all forces must always pass through the downstream extremity of the middle third of the base

(*vii*) Due to valley storage, the peak discharge of an outflow hydrograph is :

(*a*) increased (*b*) reduced
(*c*) not affected (*d*) sometimes increased and sometimes reduced

(*viii*) According to Dicken's formula for estimating floods, the peak discharge is proportional to :

(*a*) A (*b*) $A^{1/2}$
(*c*) $A^{2/3}$ (*d*) $A^{5/3}$

where A is the catchment area in km^2

Q. C. *Match the sentences of column A to that of column B*

(*i*)	*Column A*	*Column B*
	(*a*) Ryve's formula	(A) to determine average depth of rainfall
	(*b*) Theissen's polygon	(B) for estimation of peak flood
	(*c*) Lacey's theory	(C) design of alluvial channels

(*ii*)

Column A	*Column B*
(*a*) Isohyet	(A) line joining points on catchment having equal time of travel
(*b*) Isochrone	(B) lines having equal depth of evaporation
(*c*) Isopleths	(C) line joining points of equal rainfall magnitude

(*iii*)

Column A	*Column B*
(*a*) Lysimeter	(A) measurement of stage
(*b*) Current meter	(B) measurement of evapotranspiration
(*c*) Bubble gauge	(C) measurement of velocity of streamflow

(*iv*)

Column A	*Column B*
(*a*) Hydraulic jump	(A) Spillway
(*b*) Seepage	(B) Hydroelectric power project
(*c*) Penstock	(C) Earth dam

Answers to Questions of 1997 (*Winter*) Exam.

A (*i*) the quantity of water retrained by a given soil against the pull of gravity, when expressed as depth of water spread over the basin

A (*ii*) 120

A (*iii*) the rainfall rate is less than the infiltration capacity

A (*iv*) rainfall rate Vs. time

A (*v*) silt grade

A (*vi*) HFL of drain is a little bit higher than the canal bed level

A (*vii*) rate of rainfall above which the rainfall volume becomes equal to runoff volume (Note. See Eqn. 7.102)

B (*i*) –(*b*) **B (*ii*)** –(*d*) **B (*iii*)** – (a) **B (*iv*)** – (*c*)

B (*v*) – (*a*) **B (*vi*)** – (b) **B (*vii*)** –(*b*) **B (*viii*)**– (*d*)

C (*i*) – (*a*)-(*B*), (*b*)-(*A*), (*c*)-(*C*) **C (*ii*)** – (*a*)-(*C*), (*b*)-(*A*), (c)-(*B*)

C (*iii*) – (*a*)-(*B*), (*b*)-(*C*), (*c*)-(*A*) **C (*iv*)** – (a)-(*A*), (*b*)-(*C*), (*c*)-(*B*)

Hints for Solving Starred Questions of 1997 (*Winter*) Exam

Q. B (*v*) Dominant discharge = 1600 cumecs

$$\text{Max. or Peak discharge} = \frac{16}{9} \times 1600 \text{ cumecs}$$

Waterway for a bridge = Lacey's Regime Perimeter = $4.75\sqrt{Q}$

where Q is the peak design discharge

$$\therefore \text{Waterway} = 4.75\sqrt{\frac{16}{9} \times 1600} = 4.75 \times \frac{4}{3} \times 40$$

$$= 253.3 \text{ m} \cong \mathbf{250\ m} \quad \textbf{Ans.}$$

Year 1998 (*Summer*) Exam

Q. (A) *Choose the correct answer from the various alternatives for the following :* (1 × 14)

(*i*) For a crop, the consumptive use of water is equal to the depth of water :
- (*a*) transpired
- (*b*) evaporated
- (*c*) transpired and evaporated
- (*d*) used in transpiration, evaporation and evaporated from adjacent soil

(*ii*) Effective precipitation is defined as :
- (*a*) Precipitation minus the loss due to evaporation
- (*b*) Precipitation minus the loss due to infiltration
- (*c*) Precipitation occurred during the crop period
- (*d*) available water stored within the root zone

(*iii*) Irrigation canals are generally aligned along :
- (*a*) ridge line
- (*b*) contour line
- (*c*) valley line
- (*d*) straight line

(*iv*) The difference in elevation between top of dam and full reservoir level is called:
- (*a*) berm
- (*b*) free board
- (*c*) height of bank
- (*d*) wave height

(*v*) The land is said to be waterlogged, if the soil pores within :
- (*a*) a depth of 40 cm is saturated
- (*b*) a depth of 60 cm is saturated
- (*c*) root zone of crops is saturated
- (*d*) soil up to ground watertable is saturated

(*vi*) An outlet maintaining a constant discharge, irrespective of fluctuation in water levels of supplying canal and water course, is known as
- (*a*) non-modular outlet
- (*b*) semimodular outlet
- (*c*) flexible modular outlet
- (*d*) rigid modular outlet

(*vii*) A drop in canal bed is generally provided, if :
- (*a*) ground slope exceeds designed bed slope
- (*b*) design bed slope exceeds ground slope
- (*c*) ground slope is same as designed bed slope
- (*d*) none of the above.

(*vii*) Bligh's Creep theory of seepage assumes :
- (*a*) equal weightage to horizontal and vertical creep
- (*b*) more weightage to horizontal creep than vertical creep
- (*c*) less weightage to horizontal creep than vertical creep
- (*d*) loss of head follows a sine curve.

(*ix*) The structure constructed to allow drainage water to flow under pressure through an inverted syphon below a canal is called :

(*a*) syphon (*b*) superpassage

(*c*) syphon aqueduct (*d*) super aqueduct

(*x*) The meandering of river is due to :

(*a*) sediment load of streams

(*b*) discharge and hydraulic properties of stream

(*c*) erodibility of the bed and banks of streams

(*d*) all of the above.

(*xi*) According to thin cylinder theory, the volume of concrete required for an arch dam would be minimum if the central angle is

(*a*) 130°34′ (*b*) 133°34′

(*c*) 136°34′ (*d*) 139°34′

(*xii*) According to Ryve's formula for estimating floods, the peak discharge is proportional to :

(*a*) A (*b*) $A^{3/4}$

(*c*) $A^{2/3}$ (*d*) $A^{1/2}$

(*xiii*) Levees are constructed :

(*a*) parallel to river flow

(*b*) transverse to the river flow

(*c*) perpendicular to the river flow

(*d*) all the above.

(*xiv*) Hydrograph is a plot of :

(*a*) rainfall intensity against time

(*b*) discharge against time

(*c*) cummulative rainfall against time

(*d*) cummulative discharge against time

(B) Fill in the blanks in the following statements :

(*i*) If the duty for a base period of 120 days is 1500 hectares/cumec, then the delta for crop is ____ m.

(*ii*) If a river in alluvial plain has dominant charge of 1600 cumecs, the waterway for a bridge on this river should be ____ m (appx.)

(*iii*) For aone metre depth of flow in a canal, with CVR equal to 0.90, the velocity of flow according to Kennedy's theory will be ____ m/s.

Answers to Questions of 1998 (*Summer*) Exam

A (*i*)–(*d*)	*A* (*ii*)–(*b*)	*A* (*iii*)–(*a*)	*A* (*iv*)–(*b*)
A (*v*)–(*c*)	*A* (*vi*)–(*d*)	*A* (*vii*)–(*a*)	*A* (*viii*)–(*a*)
A (*ix*)–(*c*)	*A* (*x*)–(*d*)	*A* (*xi*)–(*b*)	*A* (*xii*)–(*c*)
A (*xiii*)–(*a*)	*A* (*xiv*)–(*b*)		
B (*i*)–0.69 m	*B* (*ii*)–250 m	*B* (*iii*)–0.495	

Year 1998 (*Winter*) Exam.

Q. (A) *Fill in the blanks :* (1 × 10)

(*i*) Consumptive use is defined as .....

(*ii*) The relationship between duty and delta is .....

(*iii*) Base period of an irrigated crop is the period from ..... to .....

(*iv*) Hydraulic mean depth is the ratio between ..... and .....

(*v*) Fan shaped catchment gives ..... run off than Fern leaf shaped catchment.

(*vi*) Tortuosity of a meandering river is the ratio between ..... and .....

(*vii*) Middle third rule is used in .....

(*viii*) ..... is used commonly to measure actual evapotranspiration.

(*ix*) The sequent peak algorithm is a technique used in the estimation of .....

(*x*) The surge tank is provided in the water supply line to reduce the effect of .....

Q. (B). *Choose the correct statement for the following* (1 × 10)

(*i*)* A rain-fall with an intensity of 5 mm/h is classified as

(*a*) trace (*b*) light rain

(*c*) moderate rain (*d*) heavy rain.

(*ii*) Double mass curve technique is adopted to :

(*a*) estimate the missing data.

(*b*) obtain intensities of rainfall at various durations

(*c*) check the consistency of data.

(*d*) obtain the storage capacity needed for a reservoir.

(*iii*) The drainage density is the :

(*a*) average length of streams per unit drainage are within the basin.

(*b*) stream discharge per unit drainage area

(*c*) drainage area per unit stream length

(*d*) number of streams per unit drainage area.

(*iv*) Streams that contribute to the ground water are called :

(*a*) effluent streams (*b*) ground water streams

(*c*) influent streams (*d*) perennial streams.

(*v*) The basic assumptions of unit hydrograph theory are :

(*a*) linear response and time variance

(*b*) linear response and time invariance

(*c*) non-linear response and time variance

(*d*) non-linear response and time invariance

(*vi*) The probable maximum flood is :

(*a*) an impossibly large flood discharge

(*b*) a flood discharge that is maximum over the past years.

(*c*) an extremely large but physically possible flood at the site.

(*d*) a flood with maximum probability of occurrence.

(*vii*) If γ' is the submerged unit weight of soil, ϕ is the angle of internal friction and h is the height to which the silt is deposited, they the pressure on the dam due to silt is given by :

(*a*) $P_s = \gamma' \cdot h^2 \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ (*b*) $P_s = \gamma' \cdot h^2 \cdot \left(\frac{1+\sin\phi}{1-\sin\phi}\right)$

(*c*) $P_s = \frac{1}{2} \cdot \gamma' h^2 \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$ (*d*) $P_s = \frac{1}{2} \cdot \gamma' \cdot h^2 \left(\frac{1+\sin\phi}{1-\sin\phi}\right)$

(*viii*) In cross-drainage works, the function of an inlet is to admit drain water into a :

(*a*) canal (*b*) field

(*c*) drain (*d*) none of the above

(*ix*)* The points connecting equal evapotranspiration is known as :

(*a*) isobar (*b*) isopleth

(*c*) isohyet (*d*) isovel.

(*x*) Leaching is the process by which :

(*a*) water is excessively pumped from a water-logged area.

(*b*) excess salt in the soil is pushed down by standing water above the soil.

(*c*) improving the soil structure end texture by adding suitable chemicals.

(*d*) sodium ions are removed from irrigation water.

Answers to Questions of 1998 (*Winter*) Exam

A (*i*). the amount of water used in evapotranspiration from an area under vegetation including the water used by the plants in their metabolic process for building of plant tissues.

A (*ii*). $\Delta = \frac{8.64\,B}{D}$ m where D = duty in ha/cumec

B = Base period of crop in days

Δ = Delta in m.

A (*iii*)–the time between the first watering of a crop at the time of its sowing to its last watering before harvesting.

A (*iv*)–area of canal; Wetted perimeter of canal.

A (*v*)–greater

A (*vi*)–arcual length along the river reach; direct axial length of the river reach.

A (*vii*)–the design of concrete gravity dams.

A (*viii*)–A lysimeter

A (*ix*)–minimum storage capacity of a reservoir.

A (*x*)–water hammer -

B (*i*)–(*c*) **B** (*ii*)–(*c*) **B** (*iii*)–(*a*) **B** (*iv*)–(*b*)

B (*v*)–(*b*) **B** (*vi*)–(*c*) **B** (*vii*)–(*c*) **B** (*viii*)–(*a*)

B (*ix*)–(*b*) **B** (*x*)–(*b*)

Hints for Solving Starred Questions of 1998 (*Summer*) Exam

Q. B (*i*). On the basis of its intensity, the rainfall is classified as light, moderate or heavy, as shown in table 33.1.

Table 33.1. Types of Rainfalls

S. No.	*Type of rainfall*	*Intersity of rainfall*
1	Light rain	trace to 2.5 mm/h
2	Moderate rain	> 2.5 mm/h to 7.5 mm/h
3	Heavy rain	> 7.5 mm/h

Q.B. (*ix*). As defined in the text, an *isobar* is a curve which joins the points of equal atmospheric pressure; while an *isohyet* is the one which joins the points of equal rainfall in a catchment plan.

An **isovel** on the other hand, is a contour (a curve) connecting points of equal velocities is a river cross-section; while an **isopleth** is a curve which connects the points of equal evapotranspiration. IMD has provided isopleths-map for the country, showing isopleths of annual PET (cm), wherein Delhi is found to be on the isopleth curve of 160 cm.

Year 1999 (*Summer*) Exam.

Q. (A). *Fill in the blanks of the following statements* (1 × 10)

(*i*) Percolation losses are more from a canal that is .....

(*ii*) Lacey's theory is applicable only to ..... rivers and not to .....

(*iii*) Gross command area is the sum of ..... and .....

(*iv*) Piping under an earthen dam starts developing from the ..... side of the structure.

(*v*) The loss of water during the growth of a crop is termed as .....

(*vi*) The flow occurring in a tunnel with water level above the roof at the exit end is ..... flow.

(*vii*) The drainage holes provided inside the body of a concrete gravity dam are located on the ..... side of the grout curtain.

(*viii*) The type of hydraulic structure to raise or lower the vessel travelling in a navigation canal is called a .....

(*ix*) A syphon is a cross-drainage work in which the canal is taken ..... the river.

(*x*) Repelling spurs project from the bank and form an acute angle on the ..... side.

Q. B. *Pick up the right alternative from the various choices provided in the following questions :* (1 × 10)

(*i*) For conveying the same discharge, a lined canal can have a bed slope :

(*a*) steeper than an unlined canal

(*b*) milder than an unlined canal

(*c*) same as an unlined canal

(*d*) none of the above.

(*ii*) An arch dam requires that :
- (*a*) the foundation be stronger than the abutments
- (*b*) the abutments be stronger than the foundation
- (*c*) the foundation consists of gravel and sand.
- (*d*) None of the above.

(*iii*) By increasing the length of the floor on the upstream of the weir axis :
- (*a*) the uplift pressure will increase below the floor downstream of the weir.
- (*b*) the uplift pressure will decrease below the floor downstream of the weir.
- (*c*) the uplift pressure will remain the same in both the cases
- (*d*) none of the above.

(*iv*) The nose of the divide wall of a barrage is founded on :
- (*a*) firm rock which may be at any depth
- (*b*) well foundation
- (*c*) pile foundation
- (*d*) none of the above.

(*v*) The scour depth as measured below the highest flood level of a river :
- (*a*) will be more in a bouldery river than in an alluvial river
- (*b*) will be less in a bouldery river than in an alluvial river
- (*c*) will be the same in both cases
- (*d*) none of the above.

(*vi*) In a level crossing type of cross-drainage works :
- (*a*) the canal is above the drain
- (*b*) the canal is below the drain
- (*c*) the canal and the drain are at the same level
- (*d*) none of the above.

(*vii*) The tunnels of a sediment excluder are so designed that :
- (*a*) the longest tunnel is farthest from the canal head regulator.
- (*b*) the longest tunnel is nearest to the canal head regulator.
- (*c*) all the tunnels are of the same length
- (*d*) none of the above.

(*viii*) Khosla's theory is applicable to structures founded on :
- (*a*) rock
- (*b*) any type of material
- (*c*) permeable material only
- (*d*) none of the above.

(*ix*) To minimise the head losses at the entry of a branch canal taking off from the main canal, the centre line of the branch canal should make an angle with the centre line of the main canal that is :
- (*a*) obtuse in the direction of flow
- (*b*) acute in the direction of flow
- (*c*) obtuse or acute
- (*d*) none of the above.

(*x*) For a metering flume, the crest over which the head is measured should be :

(*a*) sharp (*b*) broad

(*c*) either sharp or broad (*d*) none of the above.

Answers to Questions of 1999 (*Summer*) Exam.

A (*i*)–Unlined

A (*ii*)–stable alluvial ; rocky or boulder river stages and unstable (aggrading or degrading) alluvial rivers.

A (*iii*)–cultivable ; non-cultivable area which can be economically irrigated without considering the limitation of the quantity of available irrigation water.

A (*iv*)–downstream **A** (*v*)–evapotranspiration **A** (*vi*)–submerged

A (*vii*)–upstream **A** (*viii*)–navigation lock **A** (*ix*)–below

A (*x*)–upstream

B (*i*)–(*b*) **B** (*ii*)–(*b*) **B** (*iii*)–(*b*) **B** (*iv*)–(*b*)

B (*v*)–(*b*) **B** (*vi*)–(*c*) **B** (*vii*)–(*b*) **B** (*viii*)–(*c*)

B (*ix*)–(*b*) **B** (*x*)–(*c*)

Year 1999 (*Winter*) Exam

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) If the duty for a crop with base period of 120 days is 1500 ha/cumec, then its delta is ..... .

(*ii*) An isohyet is a line joining points of ..... .

(*iii*) The infiltration capacity of a soil ..... with time.

(*iv*) Canals taken directly from the rivers with or without head regulators are called ..... .

(*v*) When the canal with bed level higher than the high flood level in drain cross each other, then the cross drainage structure is called ..... .

(*vi*) The suction pressure on the ogee spillway is caused, when the head on spillway is ..... the design head.

(*vii*) An example of vertical lift gate is ..... .

(*viii*) Cross drainage works are not needed when canal is aligned as ..... .

(*ix*) Movement of water under or around an impervious structure is called ..... .

(*x*) The type of dam preferred for a narrow gorge with strong abutments is ..... .

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) Enveloping curve is a method to :

(*a*) determine the peak flood (*b*) determine the infiltration capacity

(*c*) estimate interception (*d*) route the flood through reservoir.

(*ii*) The maximum water application efficiency is in :

(*a*) surface irrigation (*b*) lift irrigation

(*c*) sprinkler irrigation (*d*) furrow irrigation.

(*iii*) Evaporation depends on :

(*a*) temperature (*b*) humidity
(*c*) wind velocity (*d*) all the above.

(*iv*) Canals meant for the purpose of draining off water from water-logged areas are called :

(*a*) seepage canals (*b*) laterals
(*c*) drains (*d*) water courses.

(*v*) Cross drainage work, where the bed levels of canal and drain are equal is called :

(*a*) aqueduct (*b*) level crossing
(*c*) super passage (*d*) siphon aqueduct.

(*vi*) The most economical central angle of an arch dam is :

(*a*) 93° (*b*) 123°
(*c*) 133° (*d*) 183°.

(*vii*) Radial gate is also known as :

(*a*) tainter gate (*b*) stoney gate
(*c*) drum gate (*d*) rolling gate.

(*viii*) Guide banks are provided to :

(*a*) decrease depth of flow in river (*b*) confine the width of river
(*c*) reduce the flood peak (*d*) maintain minimum depth in river.

(*ix*) River intake is usually situated on :

(*a*) convex side of curve (*b*) concave side of curve
(*c*) straight reach (*d*) downstream side of sewer outfall.

(*x*) Stability of the upstream slope of an earthen dam becomes critical for the condition of :

(*a*) reservoir full (*b*) reservoir empty
(*c*) sudden drawdown (*d*) slow filling of reservoir.

Answers to Questions of 1999 (*Winter*) Exam.

A (*i*)–69 cm *A* (*ii*)–equal rainfall *A* (*iii*)–varies
A (*iv*)–main canals *A* (*v*)–aqueduct *A* (*vi*)–more than
A (*vii*)–stoney gate *A* (*viii*)–water-shed canal *A* (*ix*)–seepage flow
A (*x*) an arch dam

B (*i*)–(*a*) *B* (*ii*)–(*c*) *B* (*iii*)–(*d*) *B* (*iv*)–(*c*)
B (*v*)–(*b*) *B* (*vi*)–(*c*) *B* (*vii*)–(*a*) *B* (*viii*)–(*b*)
B (*ix*)–(*c*) *B* (*x*)–(*c*)

Year 2000 (*Summer*) Exam.

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) Thiessen polygon method is used to find ..... .
(*ii*) Sugar-cane, according to crop period is ..... crop.
(*iii*) Base period is ..... than the crop period.

(*iv*) Weed growth in unlined canal is ....., if the velocity of water is less than 60 cm/sec.

(*v*) Fish ladder is a component part in ..... .

(*vi*) Passage provided in gravity dam to drain seepage water is called ..... .

(*vii*) If bed level of canal is at higher level than that of drain, then the cross-drainage work is called ..... .

(*viii*) Gibb's module is an example for ..... outlet.

(*ix*) Deflecting groynes are constructed on ..... bank of river.

(*x*) Water-logging, takes place due to ..... in ground water table.

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) Kharif crops are sown and harvested in the months of :

(*a*) January and May (*b*) July and January

(*c*) April and September (*d*) October and March.

(*ii*) If the pan evaporation indicates a total yearly evaporation of 1.5 m, then the annual water loss by evaporation from reservoir of water surface area of 100 km^2 will be :

(*a*) 1.5×10^3 ha-m (*b*) 1.5×10^4 ha-m

(*c*) 1.5×10^5 ha-m (*d*) 1.5×10^6 ha-m.

(*iii*) Water enters the soil at capacity rate when intensity of rainfall is

(*a*) more than the infiltration capacity of the soil

(*b*) less than the infiltration capacity of the soil

(*c*) equal to the infiltration capacity of the soil

(*d*) slightly less than the infiltration capacity of the soil.

(*iv*) With the increase in quantity of water supplied, the yield of most crops :

(*a*) continuously increases

(*b*) continuously decreases

(*c*) increases up to certain limit and then decreases

(*d*) decreases up to certain limit and then increases.

(*v*) The ordinates of unit hydrograph may be obtained by dividing the ordinates of the direct runoff hydrograph of a storm by :

(*a*) duration of unit hydrograph (*b*) storm duration

(*c*) base flow volume (*d*) direct runoff volume.

(*vi*) For triangular dam section of height 'H', for just no tension, under action of water pressure, self weight and uplift pressure, the minimum base width required with usual notation is :

(*a*) $\frac{H}{S-1}$ (*b*) $\frac{H}{(S-1)^{-1}}$

(*c*) $\frac{H}{\sqrt{S}}$ (*d*) $\frac{H}{\sqrt{S-1}}$.

(*vii*) Due to valley storage, the peak discharge of an outflow hydrograph :

(*a*) increases (*b*) reduces

(*c*) is not affected (*d*) not any of the above.

(*viii*)* The upstream face of the earth dam is considered as :

(*a*) equipotential line (*b*) stream line

(*c*) streak line (*d*) path line.

(*ix*) A super passage is a cross-drainage work provided when bed of natural drain is :

(*a*) at the same level as the canal bed

(*b*) well above the full supply level in canal

(*c*) below the full supply level of canal

(*d*) below the bed level of canal.

(*x*) River training works are seldom required in

(*a*) rocky stage of a river (*b*) meandering river

(*c*) trough stage of a river (*d*) deltaic stage of river.

Answers to Questions of 2000 (*Summer*) Exam.

A (*i*)–mean rainfall over a drainage area *A* (*ii*)–an overlapping

A (*iii*)–slightly less *A* (*iv*)–more *A* (*v*)–canal head works

A (*vi*)–drainage gallery *A* (*vii*)–an aqueduct or a syphon aqueduct

A (*viii*)–modular or rigid *A* (*ix*)–concave *A* (*x*)–rise

B (*i*)–(*c*) *B* (*ii*)–(*b*) *B* (*iii*)–(*a*) *B* (*iv*)–(*c*)

B (*v*)–(*d*) *B* (*vi*)–(*d*) *B* (*vii*)–(*b*) *B* (*viii*)–(*a*)

B (*ix*)–(*b*) *B* (*x*)–(*a*)

Hints for Solving Starred Questions

B (*viii*) Equipotential lines are the lines of equal energy, since every point on the upstream face of an earth dam will be under equal total energy (pressure head + datum head + vel. head), this line acts as an equipotential line. Similarly, the upstream floor of a weir acts as an equipotential line.

Year 2000 (*Winter*) Exam.

Q. A. Fill in the blanks in the following statements : (1 × 10)

(*i*) A groyne projecting from the river bank and with its nose pointing upstream is a ..... groyne.

(*ii*) A canal in which the amount of excavation is same as in the fill portion is called a ..... section.

(*iii*) A lined canal leading water to a power house has a slope ..... than an unlined canal for irrigation.

(*iv*) An arrangement for removal of silt that has already entered a channel is called a silt ..... .

(*v*) A cistern in a canal fall serves as a ..... .

(*vi*) The structure provided in a concrete dam to release excess water during floods is called a ..... .

(*vii*) Failure of an earth dam due to flow of water under the foundation and emerging on the downstream side is termed as a failure due to ..... .

(*viii*) A cross dreinage work in which the canal water and the drainage meet at the same level is called a ..... .

(*ix*) Crops grown during winter months and harvested in March-April are called ..... crops.

(*x*) A device to release water from a canal to the water course is called an ..... .

Q. B. *Choose the right answer from the list given below each question :* (1 × 10)

(*i*) Lacey's regime theory was developed after extensive research in :

(*a*) Israel (*b*) Indonesia
(*c*) India (*d*) Iraq.

(*ii*) A chute spillway is generally provided with :

(*a*) a weir (*b*) a barrage
(*c*) a concrete gravity dam (*d*) an earth dam.

(*iii*) A siphon spillway in a concrete gravity dam is in the shape of :

(*a*) latter U
(*b*) an inverted U
(*c*) horizontal bend through the abutment
(*d*) any straight line.

(*iv*) Energy will be required to irrigate land :

(*a*) under gravity (*b*) much above the river bed level
(*c*) in delta region (*d*) in all the above cases.

(*v*) On a meandering river, the current is strongest on :

(*a*) the concave bend (*b*) the convex bend
(*c*) both banks (*d*) the middle of the stream.

(*vi*) A barrage is a hydraulic structure provided on :

(*a*) deep narrow valleys (*b*) wide shallow valley
(*c*) rock foundation (*d*) silts, clays and gravels.

(*vii*) Conjunctive use of water means :

(*a*) use of surface water alone
(*b*) use of ground water alone
(*c*) use of pumped water alone
(*d*) combined use of surface water and ground water.

(*viii*) A straight concrete gravity dam should have :

(*a*) strong abutments (*b*) clay foundations
(*c*) strong foundations (*d*) None of the above.

(*ix*) Flexibility of an outlet is the ratio of :

(*a*) rate of change of discharge in an outlet to the normal depth in the channel
(*b*) rate of change of discharge in an outlet to the rate of change of discharge in the parent channel.
(*c*) rate of depth of crest level of outlet and FSL of channel to its full supply depth.
(*d*) head recovered to head put in.

(*x*) A canal on a watershed requires :

(*a*) a level crossing (*b*) a syphon

(*c*) a super passage (*d*) None of the above.

Answers to Questions of 2000 (*Winter*) Exam.

A (*i*)–repelling (or deflecting) *A* (*ii*)–balance

A (*iii*)–steeper *A* (*iv*)–ejector *A* (*v*)–device for energy dissipation

A (*vi*)–spillway *A* (*vii*)–piping *A* (*viii*)–level crossing

A (*ix*)–Rabi *A* (*x*)–Outlet

B (*i*)–(*c*) *B* (*ii*)–(*d*) *B* (*iii*)–(*b*) *B* (*iv*)–(*b*)

B (*v*)–(*a*) *B* (*vi*)–(*b*) *B* (*vii*)–(*d*) *B* (*viii*)–(*c*)

B (*ix*)–(*b*) *B* (*x*)–(*d*)

Year 2001 (*Summer*) Exam.

Q. A. *Fill in the blanks :* (2 × 5)

(*i*) In a hydraulic structure, the position of hydraulic gradient line is 2.0 m from the top of the impervious floor. The relative density of the material of impervious floor is 2.40. From uplift consideration, floor thickness at this location should be ..... .

(*ii*) A launching apron in a hydraulic structure provides safety against ..... .

(*iii*) In a hydro-power scheme, the pipe between surge tank and power house is known as ..... .

(*iv*) The difference in water level u/s and d/s of a bridge is known as ..... .

(*v*) Equation ..... given by Laccy is used for fixing the waterway of bridges and barrages in alluvial streams.

Q. B. *Choose the correct statement for the following :* (1 × 10)

(*i*) In a design of canal, falls are provided :

(*a*) to dissipate the energy

(*b*) when ground slope is steeper than the canal bed slope

(*c*) when the canal bed slope is steeper than the ground slope

(*d*) to reduce hydraulic losses.

(*ii*) Permeable spurs are used to :

(*a*) promote siltation along the banks of the river

(*b*) deflect the flow

(*c*) attract the flow

(*d*) contract the width of the river for navigation purposes.

(*iii*) Lacey's silt factor *f* for the average size of the river bed material of 0.30 mm is :

(*a*) 0.96 (*b*) 1.0

(*c*) 1.2 (*d*) 0.3.

(*iv*) For the estimation of flood peak, the value of Q in Dicken's formula is given by :

(*a*) $CM^{3/4}$ (*b*) $CM^{2/3}$

(*c*) $CM^{3/2}$ (*d*) $CM^{4/3}$.

(*v*) Seepage losses in a canal are more when it is :

(*a*) completely in filling (*b*) partly in filling and partly in cutting

(*c*) fully in cutting (*d*) lined.

(*vi*) Current meters are used for measurement of :

(*a*) velocity (*b*) discharge

(*c*) depth of flow (*d*) scour depth.

(*vii*) In a diversion headworks, the crest of the undersluice portion is kept :

(*a*) at the minimum river bed level

(*b*) higher than the crest level of canal head regulator

(*c*) at the same level as that of canal head regulator

(*d*) higher than the minimum bed level of the river.

(*viii*) In a syphon aqueduct, the maximum uplift pressure on the floor occurs when :

(*a*) canal is dry and drain is running full

(*b*) canal is running full and drain is dry

(*c*) both canal and drain are running

(*d*) both are dry.

(*ix*) Phreatic line in seepage analysis is defined as the line on which pressure is :

(*a*) equal to the atmosphere (*b*) greater than atmosphere

(*c*) lower than atmospheric (*d*) varying.

(*x*) Hydraulic jump occurs when flow changes from :

(*a*) super critical to sub-critical (*b*) subcritical to supercritical

(*c*) critical to turbulent (*d*) laminar to turbulent.

Answers to Questions of 2001 (*Summer*) Exam.

***A* (*i*)**–min. of 1.43 m ***A* (*ii*)**–*scouring* of river bed

***A* (*iii*)**–penstock ***A* (*iv*)**–afflux ***A* (*v*)**–$P = 4.75\sqrt{Q}$

***B* (*i*)**–(*b*) ***B* (*ii*)**–(*a*) ***B* (*iii*)**–(*a*) ***B* (*iv*)**–(*a*)

***B* (*v*)**–(*a*) ***B* (*vi*)**–(*b*) ***B* (*vii*)**–(*a*) ***B* (*viii*)**–(*a*)

***B* (*ix*)**–(*a*) ***B* (*x*)**–(*a*)

Year 2001 (*Winter*) Exam

Q. A. *Fill in the blanks :* (1 × 10)

(*i*) Lysimeter method is used for direct measurement of ..... .

(*ii*) The commonly used rain gauge in India is ..... .

(*iii*) According to Kennedy, non silting and non scouring velocity is called ..... .

(*iv*) Escope provided across the channel at its fag end to maintain required full supply level is called ..... .

(*v*) Canal should be ..... in order to reduce the chances of water-logging.

(*vi*) Arch dam transfers major port of hydrostatic load to ..... .

(*vii*) The commonly used earth dam is ..... .

(*viii*) Outlet, whose discharge depends on the difference in water levels in distributary and water course is called ..... .

(*ix*) Earthen embankments provided to confine flood water of alluvial river, without allowing to change its course is called ..... .

(*x*) Overflow spillway is also called ..... .

Q. B. *Choose the correct answer for the following :* (1 × 10)

(*i*) The best method of applying irrigation water to sandy undulating area is :

(*a*) free flooding (*b*) furrow method

(*c*) sub-irrigation (*d*) sprinkler irrigation.

(*ii*) The evaporation from an open water surface :

(*a*) depends upon the difference of saturation vapour pressure and actual vapour pressure in air

(*b*) is independent of wind velocity

(*c*) is independent of the area of water surface

(*d*) cannot be altered by any means.

(*iii*) Unit hydrograph resulting from a storm of specified unit duration and areal distribution gives :

(*a*) 1 mm direct run off (*b*) 1 cm direct run off

(*c*) 1 ha-m of direct run off (*d*) 1 million m^3 of direct run off.

(*iv*) The crest level of under sluices is :

(*a*) higher than the crest level of head regulator

(*b*) equal to the crest level of the weir

(*c*) equal to the bed level of the river channel

(*d*) equal to the crest level of head regulator.

(*v*) By providing seepage drains, the chances of water-logging :

(*a*) decrease (*b*) increase

(*c*) will not alter (*d*) None of the above.

(*vi*) Bhakra dam is a :

(*a*) non rigid gravity dam (*b*) rigid earth dam

(*c*) rigid gravity dam (*d*) rock fill dam.

(*vii*) In gorges with steep and strong side slopes, the most suitable type of dam is :

(*a*) earth dam (*b*) gravity dam

(*c*) buttress dam (*d*) arch dam.

(*viii*) The upstream face of an earth dam is :

(*a*) a flow line (*b*) an equipotential line

(*c*) a phreatic line (*d*) a stream line.

(*ix*) If the eccentricity of the resultant falls outside the middle third, the ultimate failure of dam occurs by :

(*a*) tension (*b*) crushing

(*c*) sliding (*d*) overturning.

(*x*) Spurs provided for the river training works are :

(*a*) parallel to the river banks (*b*) transverse to the river flow

(*c*) usually made of earth bends (*d*) provided to guide the flow to a bridge.

Answers to Questions of 2001 (*Winter*) Exam.

***A* (*i*)**–Actual evapotranspiration

***A* (*ii*)**–Simon's type of standard non-recording rain gauge

***A* (*iii*)**–critical velocity ***A* (*iv*)**–tail escape ***A* (*v*)**–lined

***A* (*vi*)**–abutments ***A* (*vii*)**–of homogenous type

***A* (*viii*)**–non-modular ***A* (*ix*)**–dykes (or levees) ***A* (*x*)**–ogee spillway

***B* (*i*)**–(*d*) ***B* (*ii*)**–(*a*) ***B* (*iii*)**–(*b*) ***B* (*iv*)**–(*c*)

***B* (*v*)**–(*a*) ***B* (*vi*)**–(*c*) ***B* (*vii*)**–(*d*) ***B* (*viii*)**–(*b*)

***B* (*ix*)**–(*a*) ***B* (*x*)**–(*b*)

Year 2002 (*Summer*) Exam

Q. A. *Choose the correct answer for the following :* (1 × 10)

(*i*) Float type rain gauge belongs to the type :

(*a*) recording rain gauge (*b*) Non-recording rain gauge

(*c*) May be both of the above (*d*) None of the above.

(*ii*) The length of stream per unit area is known as :

(*a*) Overflow density (*b*) Drainage density

(*c*) Stream density (*d*) Average density.

(*iii*) Which of the following is a costlier system ?

(*a*) Gravity irrigation (*b*) Lift irrigation

(*c*) Drip irrigation (*d*) Combination of (*a*) and (*b*) above.

(*iv*) Water supply system may be managed by artificial recharge of :

(*a*) Water-logged area (*b*) Aquifers

(*c*) Pond (*d*) All of the above.

(*v*) An earthen embankment running parallel to the river to protect the area behind it is called :

(*a*) Dam (*b*) Gully

(*c*) Dyke (*d*) Flood wall.

(*vi*) For medium silt whose average grain size is 0.16 mm, Lacey's silt factor is likely to be :

(*a*) 0.30 (*b*) 0.45

(*c*) 0.70 (*d*) 1.32.

(*vii*)* The discharge per metre width at the foot of a spillway is 10 m^3/s at a velocity of 20 m/s. A perfect free hydraulic jump will occur at the foot of the spillway when the tail water depth is approximately equal to

(*a*) 4.50 m (*b*) 5.00 m
(*c*) 5.50 m (*d*) 6.50 m.

(*viii*) The foundation soil under toe of a dam has a void ratio *e*. The specific gravity of the soil solids is *G*. Factor of safety against piping is to be taken as 2.5. The maximum permissible upward exit gradient is given by

(*a*) $i = 2.5\left(\frac{G-1}{1+e}\right)$ (*b*) $i = 2.5\left(\frac{1+e}{G-1}\right)$
(*c*) $i = 0.4\left(\frac{1+e}{G-1}\right)$ (*d*) $i = 0.4\left(\frac{G-1}{1+e}\right)$

(*ix*) According to Khosla's theory, if downstream sheet file is not provided, the exit gradient will be

(*a*) 0 (*b*) 1
(*c*) ∞ (*d*) 2.

(*x*) An attractive groyne is one which is

(*a*) Inclined upstream (*b*) Inclined downstream
(*c*) Normal to bank (*d*) Same as a repelling groyne.

Q. B. *Fill in the blanks*

(*i*) A hyetograph is a graph representing rainfall intensity ..... .

(*ii*) During seepage through an earthen mass, the direction of seepage ..... to the equipotential lines.

(*iii*) When a river starts meandering, the sediment carrying capacity of the river first ..... and ultimately ..... .

(*iv*) The agriculture product ..... has the maximum demand of water.

(*v*) Facility of irrigation is preferable in the region which is ..... .

(*vi*) Electrical logging method of groundwater investigations belongs to ..... method.

(*vii*) Cheapest mode of irrigation is ..... .

(*viii*) The volume of water taken into the storage of aquifer per unit surface per unit slope of head is known as ..... coefficient.

(*ix*) Empirical formulas to estimate evaporation is ..... .

(*x*) 'Afforestation' is a method of flood control by ..... .

Answers To Questions Of 2002 (*Summer*) Exam

A (*i*)–(*a*) *A* (*ii*)–(*b*) *A* (*iii*)–(*c*) *A* (*iv*)–(*b*)
A (*v*)—(*c*) *A* (*vi*)–(*a*) *A* (*vii*)–(*d*) *A* (*viii*)–(*d*)
A (*ix*)–(*c*) *A* (*x*)–(*b*) *B* (*i*)–(V_s time) *B* (*ii*)–(*is* ⊥)
B (*iii*)–decreases ; becomes constant *B* (*iv*)–Sugarcane
B (*v*)–Tropical *B* (*vi*)—resistivity surveys *B* (*vii*)–flow irrigation
B (*viii*)–storage *B* (*ix*)–Meyer formula and Rowher's formula
B (*x*)–soil and water conservation methods of flood control

Hints for Solving Storred Questions of 2002 (*Summer*) Exam.

Q .A (*vii*) $y_1 = \frac{q}{v_1} = \frac{10}{20} = 0.5$

$$y_1\, y_2\, (y_1 + y_2) = \frac{2q^2}{g}$$

or $$0.5\, y_2\, (0.5 + y_2) = \frac{2 \times 10^2}{9.81} = 20.387$$

or $$y_2\, (0.5 + y_2) = 40.77$$

or $$0.5\, y_2 + y_2^2 - 40.77 = 0$$

or $$y_2^2 + 0.5\, y_2 - 40.77 = 0$$

or $$y_2 = \frac{-0.5 \pm \sqrt{0.25 + 4 \times 40.77}}{2}$$

or $$y_2 = \frac{-0.5 \pm 12.78}{2}$$

or $$y_2 = \frac{12.28}{2} = 6.14 \text{ m}$$ (ignoring the un feasible $-ve$ sign)

Nearest given choice to this value of 6.14m is at choice (*d*); and hence choice (*d*) is to be marked as the correct answer.

Year 2002 (*Winter*) Exam

Q. A. *Match List P with that of List Q* (1 × 10)

List P	*List Q*
(*i*) Leguminous crops	(*a*) Average rainfall
(*ii*) Thiessen polygon	(*b*) Average infiltration
(*iii*) S-curve hydrograph	(*c*) Increase nitrogen content of soil
(*iv*) Ridge canals	(*d*) Summation of unit hydrograph
(*v*) Canal lining	(*e*) Non rigid dam
(*vi*) Rock fill dam	(*f*) Relieves pore pressure in earth dam
(*vii*) Toe drain	(*g*) Energy dissipator
(*viii*) Chute-spillway	(*h*) Diversion head works
(*ix*) Fish ladder	(*i*) Have no cross-drainage works
(*x*) River intake	(*j*) Reduces water-logging
	(*k*) Flow is super critical
	(*l*) Helps to draw water in controlled manner.

Q. 3. (*Choose the correct answer for the following* (1 × 10)

(*i*) Average annual rainfal at any station is mean of average annual rainfalls over a period of

(*a*) 14 years (*b*) 21 years

(*c*) 28 years (*d*) 35 years.

(*ii*) For a crop, consumptive use of water is equal to the depth of water

(*a*) transpired by the crop

(*b*) evaporated by the crop

(*c*) used in transpiration, evaporation and quantity evaporated from adjacent soil.

(*d*) transpired and evaporated by the crop.

(*iii*) Channel is said to be in regime condition, if

(*a*) discharge is constant

(*b*) silt grade and silt charge are constant

(*c*) flows uniformly in incoherent alluvium, as that transported in suspension

(*d*) All of the above.

(*iv*) Canal fall is provided, if

(*a*) ground slope exceeds the designed bed slope
(*b*) designed bed slope exceeds the ground slope
(*c*) ground slope and bed slope are almost same
(*d*) None of the above.

(*v*) When a canal is carried over a natural drain, at crossing, the structure provided is called

(*a*) syphon (*b*) aqueduct

(*c*) super passage (*d*) level crossing.

(*vi*) Silt storage in the reservoir is same as

(*a*) dead storage (*b*) live storage

(*c*) effective storage (*d*) total storage.

(*vii*) Economic height of a dam is that height corresponding to which

(*a*) amount of silt collected is minimum

(*b*) cost of dam per unit of storage is minimum

(*c*) cost of dam per unit of storage is maximum

(*d*) free board provided is maximum.

(*viii*) Outlet is said to be proportional, if its flexibility is

(*a*) equal to zero (*b*) less than unity

(*c*) equal to unity (*d*) more than unity.

(*ix*) In a barrage, crust level is kept

(*a*) low with large gates (*b*) high with large gates

(*c*) high with no gates (*d*) low with no gates.

(*x*) River training work is required, when the river is

(*a*) aggrading (*b*) degrading

(*c*) meandering (*d*) in super-critical flow.

Answers to Questions of 2002 (*Winter*) Exam

A (*i*)—(*a*)	(*ii*)—(*a*)	(*iii*)—(*d*)	(*iv*)—(*i*)
(*v*)—(*j*)	(*vi*)—(*e*)	(*vii*)—(*f*)	(*viii*)—(*k*)
(*ix*)—(*h*)	(*x*)—(*l*)		
B (*i*)—(*d*)	***B*** (*ii*)—(*c*)	***B*** (*iii*)—(*d*)	***B*** (*iv*)—(*a*)
B (*v*)—(*b*)	***B*** (*vi*)—(*a*)	***B*** (*vii*)—(*b*)	***B*** (*viii*)—(*c*)
B (*ix*)—(*a*)	***B*** (*x*)—(*c*)		

Years 2003 (*Summer*) Exam

Q. A. *Choose the correct answer for the following [use the codes given at the end of questions to indicate your answer]* (10 × 2)

(*i*) Match the following two lists :

	List I		*List II*
(*a*)	Maxmum quantity of water that a soil can retain against gravity	1	Specific yield
(*b*)	Moisture content of a soil at which the water is no longer available in sufficient quantity to sustain growth	2	Infiltration capacity
(*c*)	Volume of water that can be extracted by gravity from a unit volume of saturated earth material	3	Field capacity
(*d*)	Maximum rate at which a soil can absorb water through the surface.	4	Permanent wilting point

Codes :

	A	B	C	D
(*a*)	3	1	4	2
(*b*)	4	3	1	2
(*c*)	3	4	1	2
(*d*)	3	4	2	1

(*ii*)* Which of the following instruments are concerned with streamflow measurements ?

(1) Sounding weight (2) Echo-depth recorder
(3) Hydrometer (4) Bubble gauge
(5) Stevenson Box

Codes

(*a*) 1, 2 and 5 (*b*) 3, 4 and 5
(*c*) 1, 2 and 4 (*d*) 2 and 5.

(*iii*) Match the following two lists :

	List I (Plots of)		*List II (Known as)*
(*a*)	Accumulated precipitation Vs time in chronological order	1	Hydrograph
(*b*)	Streamflow Vs time in chronological order	2	Hyetograph
(*c*)	Rainfall intensity Vs time	3	Flow duration curve
(*d*)	Stream discharge Vs Percentage time the flow is equalled to or exceeded	4	Flow mass curve
(*e*)	Cumulative discharge Vs time in chloronological order	5	Mass curve of rainfall

Codes :

	A	B	C	D
(*a*)	2	1	5	3
(*b*)	5	1	2	3
(*c*)	5	4	2	1
(*d*)	5	1	2	4

(*iv*) A 596 km^2 catchment has a 12 hour unit hydrograph which can be approximated as a triangle. If its time base is 144 hours, its peak ordinate is

(*a*) 23 m^3/s (*b*) 2 m^3/s

(*c*) 50 m^3/s (*d*) 13 m^3/s.

(*v*) Identify the correct statement in the following :

In the analysis of seepage through an earthen dam

(1) the upstream face is an equipotential line

(2) the upstream face is a stream line

(3) The phreatic line is a stream line

(4) the phreatic line is an equipotential line

(5) the upstream face is neither a streamline nor an equipotential line.

Code

The statements

(*a*) 4 and 5 are correct (*b*) 1 and 2 are correct

(*c*) 2 and 3 are correct (*d*) 1 and 3 are correct.

(*vi*) A coffer dam is

(*a*) a kind of gravity dam

(*b*) an earthen dam of small height built to protect important areas like townships from floods

(*c*) a temporary structure constructed to exclude water from work area during construction

(*d*) an embankment built along a river to regulate the river for navigation.

(*vii*) A non-cohesive soil has a porosity of 30% and the relative density of soil particles is 2.70. The value of critical exit gradient for this soil is

(*a*) 0.81 (*b*) 1.19

(*c*) 1.00 (*d*) 1.89.

(*viii*)* Which of the following pairs are matched correctly ?

(1) Total time from sowing of crop to its harvesting ..... base period

(2) Winter crops ..... Rabi

(3) Monsoon crop ..... Kharif

(4) Total time between the first watering for preparation of land to last watering before harvesting ..... crop period.

Codes

(*a*) 1, 2, 3 and 4 are correct

(*b*) 1, 2 and 3 are correct

(*c*) 2 and 3 are correct

(*d*) 1 and 4 are correct.

(*ix*) Water-table drops by 3 m in an irrigable area of 50 hectares. If porosity and specific retention are 0.30 and 0.10, respectively, the change in storage in hectare metres is

(*a*) 15 (*b*) 30

(*c*) 45 (*d*) 60.

(*x*) Match the following two lists relating to irrigation outlets :

List I (Name of outlet)		*List II (Type)*	
(*a*)	Kennedy gauge outlet	1	Modular
(*b*)	Gibb's outlet	2	Non-Modular
(*c*)	Submerged pipe outlet	3	Semi-modular
(*d*)	Pipe outlet discharging free		
(*e*)	Open flume outlet.		

Answers to questions of 2003 (*Summer*) Exam

A (*i*)—(*c*) *A* (*ii*)—(*a*) *A* (*iii*)—(*d*) *A* (*iv*)—(*a*)

A (*v*)—(*d*) *A* (*vi*)—(*c*) *A* (*vii*)—(*d*) *A* (*viii*)—(*c*)

A (*ix*)—(*c*) *A* (*x*)—(A)—(III), (B)—(I), (C)—(II), (D)—(III), (E)—(III)

Hints for Solving Starred Questions of 2003 (*Summer*) Exam

Q. A. (*ii*) **Sounding weights.** Current meters, which are basically used to measure velocity of streams, are weighted down by *lead weights*, called **balancing weights** or *sounding weights*, to enable them to be positioned in a stable manner at the required

location in flowing water. These weights are of streamlined shape with a fin in the rear, as shown in Fig. 7.70, and are connected to the current meter by a hanger bar and pin assmebly. Sounding weights come in different sizes and the minimum weight in N is estimated as equal to $50\overline{v}.y$, where $\overline{v}$ is the average stream velocity in the vertical in m/s and y = depth of flow at the vertical in m. These wts are hence essentially required in stream flow measurements

Echo depth recorder is an automatic recording device used in current meter measuring operations to mark the various vertical sections and to know the depths at these points. This instrument can hence be useful for quick and automatic depth reading, and is essential for current meter measurements in rivers passing heavy floods, where moving boat method is used for streamflow measurements.

Stevenson box is a water-depth recorder which may be used for measuring water depth in stream flow measurements. This instrument is hence very much concerned with streamflow measurements.

A **bubble gauge** is a permanent installation made on the bank of a stream to record water depths in the river (upto the reference level at the bed of the stream). Such an installation is hence not connected with streamflow measurements, though may be useful in computing Gauge-Discharge curves.

A (*viii*) Time taken from sowing of a crop to its harvesting is actually known as the crop period, while time between first watering and last watering before harvesting is the base period. The choices given at (1) and (4) are reverse, and hence do not match.

Kharif crops include all those crops which are grown during the summer (Kharif) season between April to September. Kharif crops, therefore includes **monsoon crops**, which are grown during the period 16th june to 15th october.

Years 2003 (*Winter*) Exam

Q. 1. *Match the List P with the List Q* (1 × 10)

List P	*List Q*
(*i*) Flow net	(*a*) Piping failure
(*ii*) Irrigation capacity	(*b*) Flood estimation
(*iii*) Isohyetal method	(*c*) Duty
(*iv*) Exit gradient	(*d*) River training
(*v*) Pan method	(*e*) Seepage control
(*vi*) Spurs	(*f*) Water-logging
(*vii*) Stilling basin	(*g*) Evaporation
(*viii*) Salinity problems	(*h*) Seepage characteristic
(*ix*) Rock toes	(*i*) Average rainfall
(*x*) Unit hydrograph	(*j*) Hydraulic energy

Q. B. *Select the correct answer for the following* (1 × 10)

(*i*) The ordinate of mass curve at any time is

(*a*) the ordinate of flood hydrograph at that time

(*b*) the area under flood hydrograph upto that time

(*c*) sum of the ordinates of flood hydrograph upto that time

(*d*) average of the area under flood hydrograph upto that time.

(*ii*) Base flow is

(*a*) the flow that seeps below the base of a dam

(*b*) the ground water flow that enters the aquifer

(*c*) the ground water flow that enters the river

(*d*) seepage flow across the earth dams.

(*iii*) Levees in the river training works are constructed

(*a*) parallel to the rive flow

(*b*) across the river flow

(*c*) inclined to the bank in U/S direction

(*d*) inclined to the bank in D/S direction.

(*iv*) Upstream and downstream cut-off at the weir base are meant to control

(*a*) seepage and uplift respectively

(*b*) uplift and seepage respectively

(*c*) exit gradient and uplift respectively

(*d*) uplift and exit gradient respectively.

(*v*) Sodium-Absorption Ratio (SAR) is expressed as

(*a*) $\dfrac{Na^+}{\left[\frac{1}{2}(Ca^{++}+Mg^{++})^2\right]}$ (*b*) $\dfrac{Na^+}{\left[\frac{1}{2}(Ca^{++}+Mg^{++})^{1/2}\right]}$

(*c*) $\dfrac{Na^+}{[(Ca^{++}+Mg^{++})^{1/2}]}$ (*d*) $\dfrac{Na^+}{[2(Ca^{++}+Mg^{++})^{1/2}]}$.

(*vi*) Balancing depth of a canal is achieved when

(*a*) the depth of cutting and height of embankment are equal

(*b*) the discharge in cutting section and between the embankment section are equal

(*c*) the volume of excavated portion and the banks is the same

(*d*) the discharge carried by the canal is maximum.

(*vii*) A land is said to be water-logged

(*a*) when the moisture content of the root zones rises to the field capacity of soil

(*b*) wen water-table touches the surface

(*c*) when water ponding occurs

(*d*) when pores of the root zone get saturated to cut-off air circulation.

(*viii*) Canal bed level is + 100 m and FSL is 104.5 m. Bed level of a natural drain crossing canal is 102.0 m and HFL is 106.0 m. Suitable cross-drainage work is :

(*a*) aqueduct (*b*) super-passage

(*c*) canal syphon (*d*) level crossing.

(*ix*) The upward acceleration of dam due to seismic activity will
(*a*) decrease the base pressure
(*b*) increase the base pressure
(*c*) not affect the effective weight of dam
(*d*) increase the horizontal dynamic force.

(*x*) A canal aligned at right angles to the contours is called
(*a*) ridge canal (*b*) contour canal
(*c*) branch canal (*d*) side slope canal.

Answers to Questions of 2003 (*Winter*) Exam

A (*i*)—(*h*) (*ii*)—(*c*) (*iii*)—(*i*) (*iv*)—(*a*)
(*v*)—(*g*) (*vi*)—(*d*) (*vii*)—(*j*) (*viii*)—(*f*)
(*ix*)—(*e*) (*x*)—(*b*)
B (*i*)—(*b*) *B* (*ii*)—(*c*) *B* (*iii*)—(*a*) *B* (*iv*)—(*d*)
B (*v*)—(*b*) *B* (*vi*)—(*c*) *B* (*vii*)—(*d*) *B* (*viii*)—(*c*)
B (*ix*)—(*b*) *B* (*x*)—(*d*)

Year 2004 (*Summer*) Exam

Q1. (*i*) Match the following two lists.

List I	*List II*
A Sandy soil	1 Venturi Flume
B Hilly region	2 Drip irrigation
C Consumptive use	3 Contour farming
D Water flow measurement	4 Blaney-Criddle equation
	5 Storage capacity

Codes :

	A	B	C	D
(*a*)	1	2	3	4
(*b*)	2	3	5	1
(*c*)	3	2	5	4
(*d*)	2	3	4	1

Q2. Match the following two lists :

List I	*List II*
A Arch dam	1 Rock foundation
B Gravity dam	2 Strong abutment
C Earth dam	3 Any type of foundation
D Buttress dam	4 Multiple arch
	5 Sandy soils

Codes :

	A	B	C	D
(*a*)	2	1	3	4
(*b*)	2	3	4	5
(*c*)	1	2	4	5
(*d*)	4	2	3	1

Q3. Match the following two lists :

List I	*List II*
A Bandhara	1 Spurs
B Groynes	2 Anicut
C Weir	3 Maharastra
D Spillway	4 Ogee
	5 Punjab

Codes :

	A	B	C	D
(*a*)	2	5	4	3
(*b*)	3	4	5	1
(*c*)	3	1	2	4
(*d*)	1	2	4	3

Q4. *Which of the following pairs are correctly matched.*

1. C.D. structure, where canal bed level and drainage bed level are same – level crossing
2. C.D. structure, when it receive and discharge drainage water from a canal-Inlet and Outlet
3. C.D. structure when canal bed level is lower than drainage bed level – Aqueduct
4. C.D. structure when canal bed level is higher than drainage bed level-Super passage.

Codes :

(*a*) 1, 2, 3 ; (*b*) 1, 2, 4 ; (*c*) 1, 2, 3, 4 ; (*d*) 1, 2.

Q5. Assertion (A) : Thickness of u/s floor of weir is nominal

Reason (R) : Thickness of impervious floor is governed by net uplift pressure.

Codes :

(*a*) Both *A* and *R* are true but *R* is not correct explanation of *A*

(*b*) Both *A* and *R* are true but *R* is correct explanation of *A*

(*c*) *A* is true but *R* is false

(*d*) *A* is false but *R* is true

(*e*) Both are false.

Q6. The following structure serves as safety valve for the canal :

(*a*) head regulator (*b*) cross regulator

(*c*) escape (*d*) canal outlet

(*d*) module.

Q7. Base period of a particular crop is 120 days. If duty is 1500 ha/cumec, the delta is :

(*a*) 69 cm (*b*) 138 cm

(*c*) 35 cm (*d*) 54 cm

(*e*) 86.4 cm.

Q8. The following is *not* the basis for the design of an arch dam :

(*a*) thin cylinder theory (*b*) elastic theory

(*c*) unit column theory (*d*) trial load theory

(*e*) creep theory.

Q9. In the empirical formula, for estimating peak flow

$Q = CA^n$ match the value of n in Lists I and II

Dicken, Ryves, Inglis, Rational	List I
1, 1/2, 2/3, 3/4, 4/5	List II

Q10. The following type of lining material of canal is *not* exposed to atmosphere :

(*a*) cement concrete (*b*) tiles

(*c*) asphaltic concrete ; (*d*) bentonite clay membrane

(*e*) reinforced concrete.

Answers to Questions of 2004 (*Summer*) Exam

1. (*d*) **2.** (*a*) **3.** (*c*) **4.** (*d*)

5. (*b*) **6.** (*c*) **7.** (*a*) **8.** (*e*)

9. Dicken-3/4, Ryves-2/3, Inglis-1/2, Rational-1 **10.** (*e*).

Appendix Table A-I : Some Important Properties of Water

Temp. °C	Sp. gravity	Unit. wt. in kN/m^3 i.e. Sp. gravity × 9.807 ($S_s \cdot \rho_w$)	Vapour pressure kN/m^2	Viscosity	
				Dynamic or Absolute $kN\text{-}sec/m^2$	Kinematic $\left(\nu = \frac{\mu}{\rho}\right)$ $m^2/sec.$
0	0.99987	9.806	0.611	1.797×10^{-6}	1.78×10^{-6}
5	0.99999	9.807	0.872	1.517×10^{-6}	1.518×10^{-6}
10	0.99975	9.805	1.226	1.306×10^{-6}	1.310×10^{-6}
15	0.99907	9.800	1.703	1.126×10^{-6}	1.124×10^{-6}
20	0.99810	9.788	2.334	1.003×10^{-6}	1.009×10^{-6}
30	0.99574	9.765	4.609	0.802×10^{-6}	0.804×10^{-6}
40	0.99228	9.731	7.60	0.652×10^{-6}	0.654×10^{-6}
60	0.98338	9.644	19.34	0.470×10^{-6}	0.478×10^{-6}
80	0.97196	9.532	46.583	0.356×10^{-6}	0.366×10^{-6}
100	0.95865	9.401	101.50	0.282×10^{-6}	0.295×10^{-6}

Appended Table A-II : Other Important Properties of Water

Standard Atmospheric Pressure

$= P_{atm} = 760$ mm of Mercury

$= 1$ bar (b)

$= 1000$ m.b.

$= 101.325$ kPa*

$= 10.33$ m of water head**

Density of water (ρ) *at 4°C*

in MKS Units $= 1000$ kg/m^3

in SI units $= 1000$ kg/m^3

$^*P_{atm} = 760$ mm of Hg $= \frac{760}{1000}$ (m of Hg) $\times 13.595 \times 9.807$ kN/m^3

$\because$ Unit wt of Hg $= S_s \cdot \rho_w = 13.595 \times 9.807$ kN/m^3

$= 101.325$ kN/m^2

$= \mathbf{101.325\ kPa}$

$^{**}P_{atm}$ = m of water height (h_w) × unit wt of water

$= h_w\,(m) \times 9.807$ kN/m$^3 = 9.807\, h_w$

$\therefore \quad 101.325 = 9.987\, h_w$

$\therefore \quad h_w = \mathbf{10.33\ m.}$

Unit weight of water (γ or w) at 4°C

in MKS units = 1000 kgf/m^3, usually written as 1000 kg/m^3

in SI Units = 9.807 kN/m^3

Dynamic Viscosity of water at 4°C

in MKS Units = 1.598×10^{-4} kgf. sec/m^2

in SI Units = 1.567×10^{-3} Ns/m^2

= 1.567×10^{-2} Poise (P)

Kinematic Viscosity of water (ν) at 4°C

in MKS Units = 1.567×10^{-6} m^2/s

in SI Units = 1.567×10^{-6} m^2/s

= 1.567×10^{-6} Stoke (St)

Appendix Table A-III. Water Resources Potentials of River Basins of India

Sl. No.	River Basin	Average Annual Surface Water Availability (50% dependability)		Utilisable Surface Water (at 75% dependability) excluding ground water	Utilisable or Replenishable Ground Water	Present Exploitation (storage)	
		Water Reesources Potential b.cum (BCM)	Per Capita (m^3)	b.cum (BCM)	b.cum	Surface water	Ground water
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	(A) **Major River Basins**						
1.	Indus	73.30	1757	46.0	26.49	16.29	21.26
2.	Ganga-Brahmaputra-Meghna						
	(*a*) Ganga	525.02	1473	250.0	170.99	39.44	74.62
	(*b*) Brahmaputra	537.2	18417				
	(*c*) Barak	48.4	7646	24.0	26.55	2.33	4.74
		585.60					
3.	Godavari	110.54	2026	76.3	46.65	25.12	15.71
4.	Krishna	78.12	1312	58.0	26.41	41.80	11.91
5.	Cauvery	21.36	666	19.0	12.30	8.60	7.62
6.	Brahmani-Baitarani	28.48	2696	18.3	4.05	4.65	0.90
7.	Mahanadi	66.88	2546	50.0	16.46	12.33	3.44
8.	Pennar	6.31	648	6.9	4.93	2.65	2.27
9.	Mahi	11.02	1057	3.1	7.19	4.72	3.53
10.	Sabarmati	3.81	421	1.9	—	1.31	
11.	Narmada	45.64	2855	34.5	10.93	7.23	3.64
12.	Tapi	14.88	1091	14.5	8.27	9.41	4.30
	(B) **Composite River Basins**						
1.	Subernarekha	12.37	1392	6.8	1.82	0.67	0.42
2.	West Flowing rivers from Tapi to Tadri	87.41	3194	11.9	8.27	11.27	
3.	West Flowing rivers from Tadri to Kanyakumari	113.53	3539	24.3	8.52	10.24	6.51
4.	East Flowing rivers between Mahanadi and Pennar	22.52	919	13.1	18.22	1.60	11.66
5.	East Flowing rivers between Pennar and Kanyakumari	16.46	383	16.7	18.84	1.84	5.58
6.	West Flowing rivers of Kutch and Saurashtra including Luni	15.10	631	15.0	11.23	4.73	6.59
7.	Area of inland drainage in Rajasthan desert	negl.	—	—	—	—	—
8.	Minor rivers draining into Myanmar (Burma) and Bangladesh)	31.0	14616	—	—	0.31	—
	Total	1869.35	2214	690.3 Say 690	431.42 Say 432	**206-54***	184.70

* An additional live storage of 6.24 b.cum has been created through medium projects each having a capacity less than 10 Mcum (not included in above data) ; thus making a total storage of 212.78 b.cum ; say **213 b.cum**

Appendix Table A-IV : Fresh Water Supplies and their Storages on Different Rivers in India

Sl. No.	Name of the River Basin	Catchment area in Mha	Average annual flow in the river basins in b.cum	Live Storage Created/Likely to be Created in b.cum		
			50% Dependability	Through Completed projects (upto 2004)	Through Ongoing projects	Through projects under consideration
1	2		4	5	6	7
1.	Indus (Up to Border)	32.13	73.30	16.29	0.28	2.58
2.	(a) Ganga	86.15	525.02	39.46	21.22	30.08
	(b) Brahmaputra Barak and Others	23.61	585.60	2.33	9.35	41.26
3.	Godavari	31.28	110.54	25.12	6.21	5.84
4.	Krishna	25.89	78.12	41.80	7.74	1.31
5.	Cauvery	8.12	21.36	8.60	0.27	0.26
6.	Pennar	5.52	6.31	2.65	2.17	—
7.	East Flowing Rivers from Mahandi to Godavari and Krishna to Pennar	8.66	22.52	1.60	1.42	0.95
8.	East Flowing Rivers Between Pennar and Kanyakumari	10.01	16.46	1.84	0.07	—
9.	Mahanadi	14.16	66.88	12.33	1.87	10.09
10.	Brahamani and Baitarni	5.18	28.48	4.65	0.88	8.72
11.	Subernarekha	—	12.37	0.67	1.65	1.38
12.	Sabarmati	2.17	3.81	1.31	0.66	0.10
13.	Mahi	3.48	11.02	4.72	0.26	0.01
14.	West Flowing Rivers of Kutch, Saurashtra including Luni	32.19	15.10	4.73	0.80	2.85
15.	Narmada	9.88	45.64	7.23	16.38	0.47
16.	Tapi	6.51	14.88	9.41	0.85	0.29
17.	West Flowing Rivers from Tapi to Tadri	5.29	87.41	11.27	3.46	0.08
18.	West Flowing Rivers from Tadri to Kanyakumari	5.62	113.53	10.24	1.32	1.45
19.	Area of Inland Drainage in Rajasthan desert	6.0	Negligible	—	—	—
20.	Minor River Basins draining into Bangladesh and Burma	3.63	31.00	0.31	—	Negligible
	Total		**1869.35**	206.54	76.26	107.54

* Additional 6.24 b.cum storage is created through medium projects having live storage of less then 0.01 b.cum which have not been accounted here, thus creating a total storage of 212.78; say **213 b.cum**

Appendix Table A-V : Statewise Position of Live Storages Created and Likely to be Created in b.cum (2004)

S. No.	State	Live storage Created in in b. cum through completed projects	Live storages to be created in b.cum through ongoing projects	Live storage to be created through projects under consideration
1.	Andhra Pradesh	27.305	6.148	1.611
2.	Arunachal Pradesh	Nil	0.241	37.934
3.	Assam	0.012	Nil	0.726
4.	Bihar	1.842	0.675	5.822
5.	Chatisgarh	6.217	0.788	0.819
6.	Goa	0.044	0.217	Nil
7.	Gujarat	16.138	7.407	3.446
8.	Haryana	Nil	Nil	0.258
9.	Himachal Pradesh	13.917	0.189	0.986
10.	Jammu & Kashmir	Nil	0.094	1.832
11.	Jharkhand	2.472	6.878	0.476
12.	Karnataka	33.631	1.413	0.066
13.	Kerala	5.384	1.336	1.686
14.	Madhya Pradesh i/c Chhatisgarh	17.156	16.776	7.335
15.	Maharashtra	25.523	13.243	0.763
16.	Manipur	0.397	8.450	Nil
17.	Meghalaya	0.698	Nil	0.516
18.	Mizoram	Nil	0.663	1.561
19.	Nagaland	1.220	Nil	0.526
20.	Orissa	17.225	1.998	21.100
21.	Punjab	2.369	Nil	Nil
22.	Rajasthan	8.285	1.426	1.807
23.	Sikkim	Nil	Nil	0.001
24.	Tamil Nadu	6.501	0.068	0.013
25.	Tripura	0.312	Nil	Nil
26.	Uttranchal	3.056	5.341	154.00
27.	Uttar Pradesh	15.345	2.712	18.407
28.	West Bangal	1.475	0.184	Nil
29.	Andaman & Nicobar island (U.T.)	Nil	Nil	Nil
30.	Chandigarh (U.T.)	Nil	Nil	Nil
31.	Dadar Nagar Haveli (U.T.)	Nil	Nil	Nil
32.	Daman & Diu (U.T.)	Nil	Nil	Nil
33.	N.C.T of Delhi	Nil	Nil	Nil
34.	Lakshdeep (U.T.)	Nil	Nil	Nil
35.	Pondicherry	0.014	Nil	Nil
	Total	206.548 Say 206.54 b.cum*	76.257 Say 76.26 b.cum	107.545 Say 107.54 b.cum

* An additional live storage capacity of 6.24 b.cum is estimated to have been created through medium projects, each having capacity of less than 0.010 b.cum (which are not included here, thus making total storage capacity of **212.78 ; say 213 b.cum** in completed projects.

Appendix Table A-VI : Levels/Capacities of Important Dam Reservoirs of India

S. No.	Name of Reservoir	Location	Dead Storage level in m	Full reservoir level in m	Capacity at Dead Storage level in M.cum	Capacity at Full Reservoir level in M. cum
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.	Nagarjunasagar	A.P.	149.05	179.83	4719	11561
2.	Sriram Sagar	A.P.	324.31 (MDDL)	332.54	849	3454
3.	Srisailum	A.P.	214.88	269.75	434	8722
4.	Ukai	Gujarat	82.30	105.16	1411	8511
5.	Sabarmati	Gujarat	170.90	189.58	54	908
6.	Kadana	Gujarat	99.08	127.71	340	1542
7.	Krishnarajasagar	Karnataka	18.29	38.04	125	1400
8.	Linganamakki	Karnataka	527.30	554.43	295	4497
9.	Tungabhadra	Karnataka	472. 44	49.74	2	3429
10.	Ghataprabha (Hidkal)	Karnataka	629.11	662.95	60	1440
11.	Malaprabha	Karnataka	619.35	633.83	96	1068
12.	Kabini	Karnataka	674.57	694.33	100	553
13.	Bhadra	Karnataka	30.48	56.70	241	26
14.	Idukki	Kerala	681.23	732.43	292	997
15.	Gandhi Sagar	M.P.	381.00	399.90	586	7413
16.	Tawa	M.P.	334.24	355.40	263	2312
17.	Mahandadi	M.P.	336.21	348.70	144	811
18.	Jayal wadi	Maharashtra	452.17	463.91	352	2909
19.	Khadakvasla	Maharashtra	574. 24	582.47	0	86
20.	Koyna	Maharashtra	609.60	657.91	120	2797
21.	Hirakud	Orissa	179.83	192.02	2318	8146
22.	Machkund	Orissa	818.35	838.16	78	971
23.	Balimala	Orissa	438.91	462.08	934	3610
24.	Salandi	Orissa	50.30	80.30	8	566
25.	Gobind Sagar	H.P.	445.62	515.11	95	9351
26.	Pong Dam	H.P.	384.00	436.70	1288	8579
27.	Rana Pratap Sagar	Rajasthan	342.90	352.91	1332	2905
28.	Lower Bhawani	Tamil Nadu	248.41	280.22	0	929
29.	Mettur	Tamil Nadu	204.22	240.79	62	2647
30.	Vaigai	Tamil Nadu	257.56	279.20	1	193
31.	Parambikulam	Tamil Nadu	534.31	556.26	125	505
32.	Adiyar	Tamil Nadu	283.46	320.04	0	109
33.	Matatila	U.P.	295.65	308.46	50	883
34.	Ramganga	U.P.	323.00	365.30	395	2448
35.	Rihand	U.P.	236.22	268.22	1640	10607
36.	Mayurkshi	West Bengal	106.38	121.31	68	617
37.	Kangsabati	West Bengal	120.43	134.14	138	52
38.	Maithon	DVC	132.59	146.30* 152.40	165	1275
39.	Panchet Hill	DVC	119.48	120.47*	170	1475
40.	Konar DVC	DVC	410.57	427.94	61	336
41.	Tilaiya	DVC	363.32	372.47	75	394

*In case of Maithon and Panchet reservoirs, F.R. levels are given without and with flood storage, both.

Appendix Table A-VII : The Highest Dams of the World

S. No.	*Name of the Dam*	*Country where situated*	*Type of Dam*	*Height in metres*	*Year in which completed*
1	Jafar Masadi dam (Sahid Yaqobi)	Iran	Earth dam	611	1996
2	Rogun dam	Russia	—do—	335	
3	Nurek dam	Russia	—do—	300	1961
4	**Grand Dixence dam**	Switzerland	Gravity dam	285*	1961
5	Inguri dam	Russia	Arch dam	272	1980
6	Vaiont dam	Italy	Arch dam	262	1961
7	Chicoasen dam	Mexico	Earth/Rockfill dam	261	1980
8	Kambaratinish dam	Russia	—do—	255	—
9	Mauvoisin dam	Switzerland	Erch dam	250	1990
10	Guavio dam	Colombia	Earth/Rockfill	250	1990
11	Sayano- Shushensk dam	Russia	Arch/gravity	245	1989
12	Mica dam	Canada	Earth/Rockfll	242	1973
13	Chivor dam (La Esmeraisa)	Colombia	Rockfill	237	1975
14	Oroviolle dam	USA	Earth dam	235	1963
15	Chirkey dam	Russia	Arch dam	233	1975
16	**Bhakra dam**	India	Gravity dam	226**	1963
17	Luzzone dam	Switzerland	Arch dam	225	1998
18	Hoover dam	USA	Arch/gravity dam	223	1936
19	Contra dam	Switzerland	Arch dam	220	1965
20	Mrantinje dam	Yogoslavia	Arch dam	220	1976
21	Dworshak dam	USA	Gravity dam	219	1973
22	Glen Canyan dam	USA	Arch dam	216	1964
23	Toktogul dam	Russia	Gravity dam	215	1978
24	Daniel Johnson dam	Canada	Multi arch dam	214	1968
25	Manicouagan 5PA dam	Canada	Multi arch dam	214	1989

* Grand Dixence is at present, the highest existing concrete gravity dam of the world.

** Bhakra dam is the highest concrete gravity dam of India, and is the second highest such dam in the world.

Appendix Table A-VIII : Largest Dams of the World

S. No.	Name of the Dam	Type of Dam	Country where Situated	App. Vol. of Fill Material in M. cum.	Year in which completed
1	Katum dam	Earth/Rock fill	Russia	4742	—
2	635 dam	Earthfill	China	458	2000
3	Ali E-delvari (Jarreh) dam	Arch	Iran	395	—
4	Sihwaho dam	Barrage Mobile (Moveable dam)	Korea (Republic)	342	1996
5	Talequan dam	Earth	Iran	329	—
6	Chapeton dam	Gravity	Argentina	296	—
7	Youngamho dam	Gravity	Korea (Republic)	245	1993
8	Penzadahe Khurded dam	Earth	Iran	175	—
9	Gumganho	Gravity	Korea (Republic)	138	1990
10	Tarbela dam	Earth/Rockfill	Pakistan	129	1976
11	Kambaratunsk	Gravity	Russia	112	—
12	Fortpeck dam	Rockfill	USA	96	1957
13	Cipasang dam	Earth/Rockfill	Indonesia	90	—
14	Ataturk dam	Rockfill	Turkey	85	1992
15	Yacyreta Apipe dam	Gravity	Argentina	68	—
16	Lee dam	Earthfill	Zimbabwe	66	—
17	Gardiner dam	Earthfill	Canada	65	1968
18	Afslulidijk	Earth	Netherland	63	1932
19	San Luis dam	Rockfill	USA	59	1967
20	Nurek dam	Earthfill	Russia	58	1980
21	**Nagarjuna Sagar dam**	Masonry gravity	India	56**	1976
22	Xiaolangdi dam	Rockfill	China	52	2002
23	Garrison dam	Rockfill	USA	51	1956
24	Oostersch-idekerng dam	Gravity	Netherland	50	1986
25	Cochiti dam	Rockfill	USA	50	1975

*Maximum Volume

**Nagarjuna Sagar dam across Krishna river is the largest dam of India, and is the highest masonry dam of the world (124.7 m high). As against this, Hirakud dam across Mahanadi river is the longest earthen dam of the world (25.3 km long).

Appendix Table A-IX : Useful Conversions Between Different Units

Length

1 metre	= 3.281 ft ;	1 ft	= 0.3048 m
1 km	= 0.622 mi ;	1 mi	= 1.6093 km
1 cm	= 0.394 in ;	1 in	= 2.54 cm
1 mm	= 1000 μ (micron)	1 micron (μ)	= 10^{-6} m

Area

1 m^2	= 10.761 ft^2 ;	1 ft^2	= 0.0929 m^2
1 km^2	= 10^6 m^2 = 100 ha	1 mi^2	= 2.59 km^2
	= 0.386 mi^2 = 247 acres		= 251 ha
			= 640 acres
1 ha	= 10^4 m^2	1 acre	= 4840 sq. yards
	= 2.47 acres ;		= 0.4047 ha

Volume and Discharge

1 m^3	= 35.307 ft^3 ;	1 ft^3	= 0.02832 m^3
1 ha.m	= 8.13 acre ft.	**1 acre ft**	= 43560 ft^3
			= 1233.6 m^3
			= 0. 123 ham
			= $\frac{1}{2}$ **cusec day**
1 m^3	= 1000 litres (L)	**1 gallon** (*i.e.* imp gallon)	= 4.546 litres
1 L	= 1.76 pints (imp)		
	= 0.22 gallon (imp)	1 U.S. gallon	= 3.785 litres.
1 cumec	= 35.307 cusec ;	1 cusec	= 0.02832 cumec

or 1 cusec = 28.32 L/s

or 1 cusec = 2.447 MLD (million litre per day)

or 1 cusec = 0.538 MGD (million gallon per day)

1 MGD = 1.86 cusec.

Force

1 kg wt	= 9.807 N (Newton) :	1 N = 0.102 kg wt
1 gm wt	= 980.7 dynes	
1 lb wt	= 32.2 poundals	
1 kg wt	= 2.204 lb wt.	

1 kg (w) = 9.807 N = 2.204 lb (wt) = 9.807×10^5 dynes

or

1 N = 0.102 kg (wt) = 0.2247 lb (wt) = 10^5 dynes

1 poundal = 138.26 dynes = 14.102 gm (wt).

Pressure

Units : N/m^2 = Pascal (*Pa*) ; kg/m^2 (*i.e.* kgf/m^2) : lb wt/ft^2

1 kg/m^2	= 9.807 N/m^2	1 P_a	= 1 N/m^2 = 0.102 kg/m^2
	= 9.807 P_a	1 kP_a	= 102 kg/cm^2

1 kg/cm^2 = 10^4 kg/m^2 100 kp_a = 10,200 kg/m^2
= 1.02 kg/cm^2

1 Atmosphere
= 760 mm of Hg
= 1 bar = 1000 mb
= 101.325 kPa = 1.033 kg/cm^2
= 10.33 m water head

1 kg/cm^2 = 98.07 kP_a = 10 m of water	100 kP_a = 1.02 kg/cm^2 = 10.2 m of water

1 lb/ft^2 = 47.88 k.P_a 100 k.P_a = 2.089 lb/ft^2
= 16.02 ft of water = 33.45 ft of water
1 psi = 1 lb/in^2 = 0.0703 kg/cm^2
1 lb/ft^2 = 4.882 kg/m^2 1 kg/cm^2 = 14.22 psi
1 kg/m^2 = 0.205 lb/ft^2

Density (ρ)

1 kg/m^3 = 0.0624 lb/cft ; 1 lb/cft = 16.02 kg/m^3

Unit wt. (γ or w = ρ. g.)

Units. N/m^3, kg/m^3 (*i.e.* kg-f/m^3) ; lb wt/ft^3

1 kg/m^3 (*i.e.* kg wt/m^3) 1 N/m^3 = 0.102 kg/m^3
= 9.807 N/m^3

Engery

Units : Joule (N.m) ; kg (wt.) m; lb (wt). m

1 Joule (j) = 1 Nm
1 kg (wt) m = 9.807 N.m
i.e. 1 kg (wt) m = 9.807 J ; 1 J = 0.102 kg (wt) m
1 ft lb (wt) = 0.138 kg (wt) m = 1.356 J
1 kg (wt) m = 7.23 ft lb (wt) = 9.807 J
1 J = 0.102 kg (wt) m = 0.737 ft lb (wt)

Power

Units : J/s ; kgm/sec ; ft lb/sec

1 J/s = 1 watt.
1 J/s = 1 N.m/s = 1 watt = 0.102 kg (wt) m/sec = 0.737 ft lb (wt)/sec
1 HP (fps) = 550 ft lbs/sec = 746 J/s = 746 watts.
1 HP (metric) = 75 kg (wt) m/sec = 75 × 9.807 = 735 J/s = 735 watts.

Dynamic Viscosity (μ or η)

Units : $N.s/m^2$; $kgf.sec/m^2$; dyne sec/cm^2 (Poise)

1 $N{\cdot}s/m^2$ = 1 Pa.s = 10 Poise = 0.102 kgf sec/m^2

1 dyne. sec/cm^2 (Poise) = 0.0102 kgf sec/m^2

Kinematic Viscosity (v)

Units : m^2/s ; m^2/sec ; cm^2/sec (Stoke)

1 $m^2/s = 10^4$ $cm^2/sec = 10^4$ Stoke (S)

1 centi stoke (CS) $= \frac{1}{100} S$

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IRRIGATION ENGINEERING AND HYDRAULIC STRUCTURES

Irrigation is the science of providing water to the lands in accordance with the requirements of the crops grown therein. It has been found to increase the productivity of rain-fed lands by as much as five times in a tropical country like India.

The annual fresh water supply (runoff) in India is 1869 b.cum. However, the present utilisation (1997) is only 605 b.cum for all its uses. Out of this, about 500 b.cum, *i.e.* about 83%, is being used for irrigation alone. Even with such a huge percentage of utilisation of water for irrigation, we are just able to irrigate only about 40% of our gross cropped area.

Additional waters are, thus, required to be developed for bringing additional cropped area under irrigation, to increase our food and fibre production. *The techniques which are employed for development and management of water resources become vitally important in this context, and have been lucidly explained in this popular book.*

The prestigious UPSC competitions continue to carry questions from this book year after year.

The present edition has further been revised and updated with rewriting of certain articles and with addition of certain new topics and numerical examples. The question bank containing questions of Engineering Services and Civil Services Exams. has also been updated.

AUTHOR'S PROFILE

The author, Mr. Santosh Kumar Garg, is a veteran in the field of civil engineering and has authored eight widely acclaimed books. His books on **Environmental Engineering Vol. I & II** as well as on **Irrigation Engineering and Hydraulic Structures** are being followed in almost all the engineering colleges in India for the last more than 18-20 years. His three new publications titled **Geotech Engineering, International and Interstate River Water Disputes,** and **Physical and Engineering Geology** have also gained a lot of popularity within a short span of their publications.