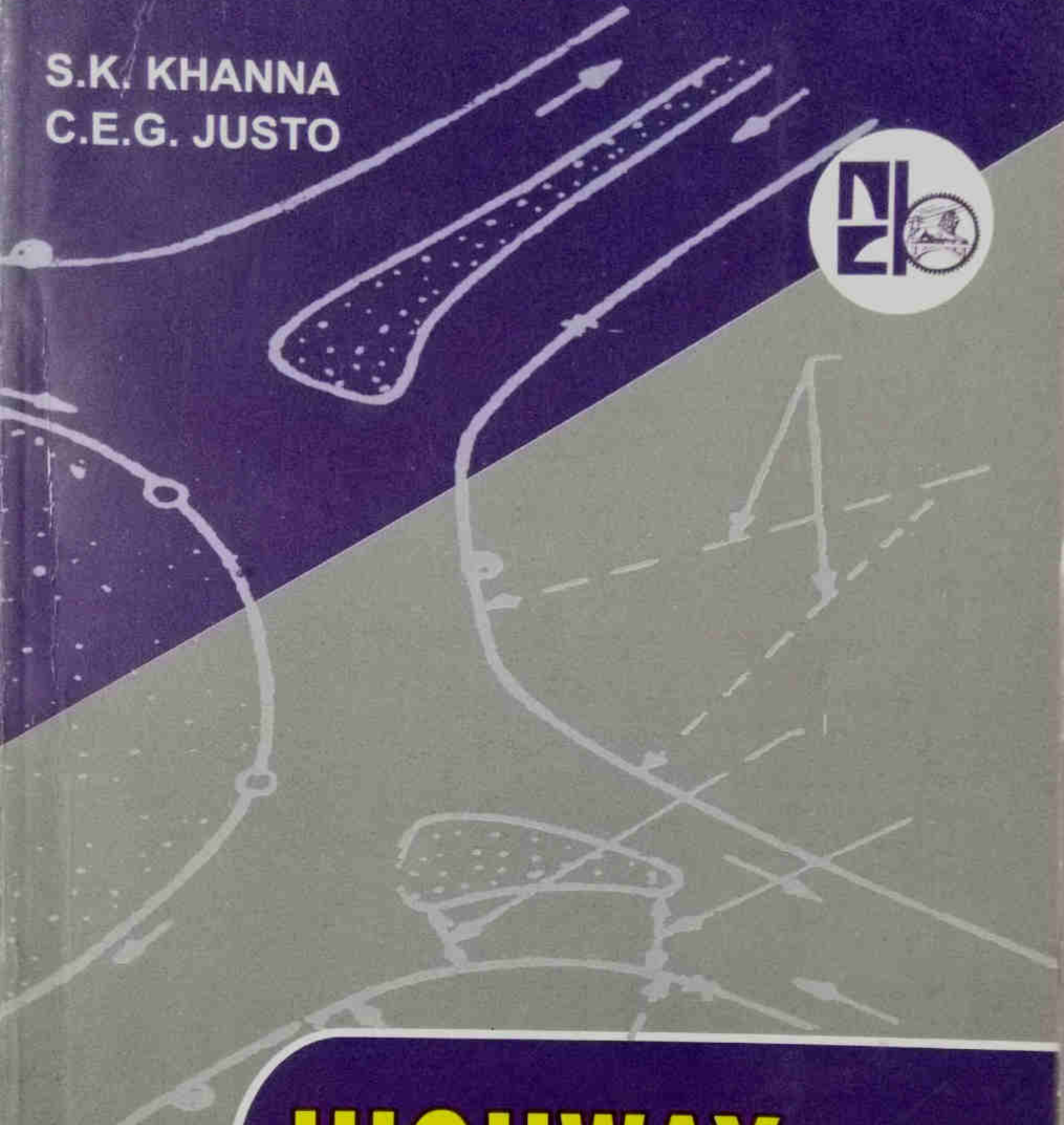
S.K. KHANNA
C.E.G. JUSTO



HIGHWAY ENGINEERING

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Edited by
Mrs. Indira Khanna & Mrs. Lalitha Justo

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PREFACE TO THE FIRST EDITION

The basic concept of highway engineering have been considerably changed in the last two decades. In the past, the subject was essentially based on the empirical practices and *thumb rules*. The design and construction thereby were done mainly based on the experiences of individual engineers. Like in other countries, in India also several organisation namely, *Central Road Research Institute, Ministry of Transport (Roads Wing), P. W. D. Research Stations* and some educational institutions have contributed a great deal to the knowledge and newer concepts in this field of engineering. All these contributions have given the subject a *scientific outlook*. The authors have been associated with the teaching and research in this discipline at the *University of Roorkee* for the past several years and as a result of this experience this book embodies the latest know-how on the subject.

It has been the objective of the authors to prepare this book introducing the subject of highway engineering to the student in a systematic manner covering the latest knowledge. The treatment of the subject is fully in *metric system*. Besides being useful for the Civil Engineering students in general, it is hoped that the book should also fulfill the need of the students of those institutions where this subject is being taught at an advanced level for the final year degree class as an optional paper or for the preparation of a complete *Highway Project*.

In preparing this book, the authors have liberally drawn information from a number of publications, particularly, of *Indian Roads Congress*. The authors feel that the book should serve the purpose of a standard text book for the undergraduate students of Civil Engineering for almost all the teaching institutions in the country.

It is possible that some errors might have been left unnoticed while the book was in print. The authors would sincerely welcome the constructive criticism for improving the book for the subsequent edition.

January 26, 1971
Roorkee

S. K. Khanna
C. E. G. Justo

PREFACE TO THE NINTH EDITION

The book has been revised in general and efforts have been made to improve the presentation. Suggestions and comments on the contents and subject matter received from the students, teachers and professionals will be considered by the authors during further revision of the book.

S. K. Khanna
C. E. G. Justo

March, 2011

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Chapter 1
Introduction

1.1 IMPORTANCE OF TRANSPORTATION

1.1.1 Role of Transportation

Transportation contributes to the economic, industrial, social and cultural development of any country. Transportation is vital for the economic development of any region since every commodity produced whether it is food, clothing, industrial products or medicine needs transport at all stages from production to distribution. In the production stage, transportation is required for carrying raw materials like seeds, manure, coal, steel etc. In the distribution stage, transportation is required from the production centres viz.; farms and factories to the marketing centres and later to the retailers and the consumers for distribution. The inadequate transportation facilities retard the process of socio-economic development of the country. The adequacy of transportation system of a country indicates its economic and social development.

1.1.2 Economic Activity and Transport

The economic activities are the processes by means of which the products are utilized to satisfy human wants. Two important factors well known in economic activity are :

(i) Production or *supply* and

(ii) Consumption for human wants or *demand*.

Man and his products are thus not bound to his local surroundings. The importance of transportation in economic activity is to be found in its effects on both human wants for goods and satisfaction through production and distribution. While discussing the general effects of transportation, it may be said that the increased productivity and its efficient transportation can lower the cost of the products. The transportation cost is always an influencing factor on consumer price of commodities.

1.1.3 Social Effects of Transportation

Progress follows the lines of transportation. Population have always settled along the river shores, road sides and near railway stations. In the present concept of transportation network, this kind of *ribbon development* is greatly discouraged. Attempts are being made to decentralize the population centres away from the sides of the main transportation routes. Thus town planning patterns are rapidly changing. To avoid congestion around the populated areas suburban living and industrial enterprises are developing. These suburbs and satellite towns, acting as counter-magnets should be linked up with rapid transit systems. The various social effects of transportation may be further elaborated as follows :

(a) *Sectionalism and transportation* : Improved transportation has important implication in reducing sectionalism within the country and also outside the country. Under-developed colonies and tribes are improving their living conditions since the distances have apparently been reduced with reduction in travel time. More-frequent travels in other parts of the country and outside the country tend to increase knowledge of the people from other sections of the society. The international understanding for better peace and order also improves with efficient network of transportation.

(b) *Concentration of population into urban area* : The improved transportation network brings prosperity to the urban population. The prosperity and employment opportunities of urban area attract the population from other areas resulting in enhanced economic activities. Adequate mass transportation facilities are needed to cater the internal movements in urban area such as daily movements to and from factories, offices, schools, hospitals and other social needs. Efficient rapid transit facilities are necessary for sub-urban and inter-city long distance travel for business needs, social visits and tourist activities. This also encourages the people to live in places away from their work centres, thus helping to decrease the growth of slums in urban areas. In general the transportation facilities are essential for the well being of the community.

(c) *Aspect of safety, law and order* : Transport facilities are essential for rushing aids to areas affected by an emergency. To maintain law and order at home, it is required to have an efficient system of transport network. To defend the territory of the country against the external aggression and to guard the borders with the foreign territories, transport facilities are needed connecting the farthest border area from the head quarters or capitals. At times, this alone may be a sufficient reason to develop a transport network which may not involve any economic and social benefit directly.

All the advantages of transportation may now be summarized :

(i) Transportation is for advancement of the community.

(ii) Transportation is essential for the economic prosperity and general development of the country, and

(iii) Transportation is essential for strategic movement in emergency for defence of the country and to maintain better law and order.

1.1.4 Role of Transportation in Rural Development

With over 75 percent of the population of the country living in the villages, the development in urban centres alone do not indicate the overall development of the country. Only with the improvement in transportation facilities in rural areas, there could be faster development of the rural centres. The fertilizers and other inputs for agriculture and cottage industries could reach the rural population easily and similarly the products

can be sold at the nearest marketing centres for more remunerative price resulting in faster economic growth and decreased wastage. With improved facilities for education, health care and other social needs in the villages, the urge for the migration to urban centres decreases, thus helping in balance development of the country as a whole.

1.2 DIFFERENT MODES OF TRANSPORTATION

Three basic modes of transport are by land, water and air. Land has given scope for development of road and rail transport. Water and air have developed waterways and airways, respectively. The roads or the highways not only include the modern highway system but also the city streets, feeder roads and village roads, catering for a wide range of road vehicles and the pedestrians. Railways have been developed both for long distance transportation and for urban travel. Waterways include oceans, rivers, canals and lakes for the movement of ships and boats. The aircraft and helicopters use the airways.

Apart from these major modes of transportation, other modes include pipe lines, elevators, belt conveyors, cable cars, aerial ropeways and monorails. Pipe lines are used for the transportation of water, other fluids and even solid particles.

The four major modes of transportation are :

(i) Roadways or highways

(ii) Railways

(iii) Waterways

(iv) Airways

The transportation by air is the fastest among the four modes. Air travel also provides more comfort apart from saving in transportation time for the passengers and the goods between the airports. Transportation by water is the slowest among the four modes; but this mode needs minimum energy to haul unit load through unit distance. The transportation by water is possible between the ports on the sea routes or along the rivers or canals where inland transportation facilities are available.

The transportation along the railway track could be advantageous by railways between the stations both for the passengers and goods, particularly for longer distances. These railway tracks could serve as arteries for transportation by land and the roads could serve as feeder system for transportation to the interior parts and to the intermediate localities between the railway stations. The energy requirement to haul unit load through unit distance by the railway is only a fraction (one fourth to one sixth) of that required by road. Therefore full advantage of this mode should be taken for the transportation of bulk goods along land where the railway facilities are available.

The transportation by road is the only mode which could give maximum service to one and all. This mode has also the maximum flexibility for travel with reference to route, direction, time and speed of travel etc. through any mode of road vehicle. It is possible to provide door to door service only by road transport. The other three modes, viz., airways, waterways and railways have to depend on transportation by roads for the service to and from their respective terminals, airports, harbours or stations. The road net work is therefore needed not only to serve as feeder system for other modes of transportation and to supplement them, but also to provide independent facility for road travel by a well planned net work of roads throughout the country.

Though co-ordination between different modes of transportation is desirable, it is also necessary to permit a healthy competition between the different modes, as each one has widely varying characteristics.

The branch of transportation engineering which deals with the planning, design, construction and maintenance of airports and other facilities for the operation of aircrafts is termed Airport Engineering. The development of terminal facilities for ships and boats, their harbouring, repairs etc. are covered under Harbour Engineering. The design and laying of railway tracks and yards, their maintenance and the safety and control of train movements are dealt in Railway Engineering. The planning, design, construction and maintenance of road and roadway facilities to cater to the needs of road traffic are covered under Road Engineering or Highway Engineering.

1.3 CHARACTERISTICS OF ROAD TRANSPORT

It is an accepted fact that of all the modes the transportation, road transport is the nearest to the people. The passengers and the goods have to be first transported by road before reaching a railway station or a port or an airport. The road network alone could serve the remotest villages of the vast country like ours.

The characteristics of road transport are briefly listed here.

(i) Roads are used by various types of road vehicles, like passenger cars, buses, trucks, two and three wheeled automobiles, pedal cycles and animal drawn vehicles. But railway tracks are used only by rail locomotives and wagons, waterways are used by only ships and boats.

(ii) Road transport requires a relatively small investment for the government. Motor vehicles are much cheaper than other carriers like rail locomotives and wagons, water and air carriers. Construction and maintenance of roads is also cheaper than that of railway tracks, docks, harbours and airports.

(iii) Road transport offers a complete freedom to road users to transfer the vehicle from one lane to another and from one road to another according to the need and convenience. This flexibility of changes in location, direction, speed and timings of travel is not available to other modes of transport.

(iv) In particular for short distance travel, road transport saves time. Trains stop at junctions and main stations for comparatively longer time.

(v) Speed of movement is directly related with the severity of accident. The road safety decreases with increasing dispersion in speed. Road transport is subjected to a high degree of accidents due to the flexibility of movements offered to the road users. Derailment of railway locomotives and air crash of air planes are also not uncommon. They are in fact more disastrous.

(vi) Road transport is the only means of transport that offers itself to the whole community alike.

1.4 IMPORTANCE OF ROADS IN INDIA

1.4.1 Significance of Planned Road Network

It may be said that deficiency in the road development in India has contributed greatly to the set-backs in agricultural, commercial and industrial sectors. It is essential to

provide roads links between the villages and market centres. The prosperity around the urban areas alone do not reflect the economic and living condition of the people of our country as a whole. Overall economic progress can be achieved, only if reasonably adequate transport facilities are made available between the villages and other district head quarters and commercial centres. The road networks have also to be supplemented with *Express ways* to keep pace with the requirement of uninterrupted movement of fast vehicles along the arterial roads. In general, developing countries have to raise their transportation systems to a higher level both in terms of length and quality so as to meet the demand which is being generated by the development plans. Also road development generates considerable employment potential. It is estimated by Planning Commission and the National Council of Applied Economic Research that Rs. 1,000 crores invested in roads would yield employment for six million persons.

It has been shown that a paved surface in reasonably good condition can contribute to 15 to 40 percent savings in vehicle operation cost. This is very significant from the point of view of energy crisis and conservation of petroleum fuel. Thus it is all the more important to construct and maintain road pavements in good condition. Revenue from the road transport in India has been much higher than the investment made on road development plans. Out of the estimated revenue of Rs. 10,000 crores from the road sector during the sixth five year plan 1980-85, only Rs. 3439 crores was provided for roads during the plan period. During the seventh plan Rs. 6,000 crores was spent for road development in the country. There is a great need and considerable scope for higher investments for development of road transportation in India.

1.4.2 Requirement of Rural Road Development

There are 5.76 lakhs villages in India. Of these only about 57 percent of villages with population about 1500, 36.3 percent villages with population 1000 to 1500 and 23 percent villages with population less than 1000 were connected with all-weather roads by 1980-81. Realising the urgency of developing the village roads, these have been treated as a part of minimum needs programme since the fifth five-year plan.

It is estimated that Rs. 11,000 crores (as per 1980 cost estimates) will be needed to provide all weather motorable road network so that on the average any village is not more than 1.6 km from a road. During the sixth five year plan (1980-85) out of the provision of Rs. 3439 crores for the road development and maintenance, Rs. 1165 crores was set apart for the rural roads under the minimum needs programme. The states are also actively engaged in the development of rural roads. For example, a master plan for rural roads has been prepared for Karnataka State at an estimated cost of Rs. 949 crores.

1.4.3 Comparative Study of Road Statistics

The road statistics give an idea of the stage of road development of a country. The road statistics may be presented on the basis of area of the country or the population. The road network of this country has to be considerably increased during the development plans. The poor state of road development in India in the past may be due to the following reasons :

(i) There was no planned development of roads in the country upto the initiation of Nagpur Road Plan in the year 1943. Only during the five-year plans since 1951, the development works were speeded up.

(ii) The investment even today on the road development programme is much lower than the revenue from the road transport.

(iii) Poor economic conditions of the vast majority of the population in villages prohibit the owning of private vehicles and discourage the use of transport.

There has been a noteworthy progress in road development in India during the post-independent period, especially since the start of the five year plans in the year 1951. The total road length by the year 1951 was only 3,97,600 km, whereas the road length achieved by 1981 was 15,02,700 km. The density of roads in km per 100 sq. km area of the country by the year 1951 was about 12 which was increased to 21 by the year 1961, 34 by 1972 and 46 by 1981. The road density values of some of the developed and developing countries by the year 1981 are given in Table 1.1 for comparison.

Table 1.1 Road Density Values in Different Countries by the Year 1981

Country	Road density in km per 100 sq. km area	Country	Road density in km per 100 sq. km area
Afghanistan	3	New Zealand	35
Egypt	3*	India	46*
Nepal	5*	U.S.A.	68
Australia	11	Hungary	94
Nigeria	12*	Italy	97
Pakistan	13*	France	146
Malaysia	14*	Great Britain	153
Brazil	16	Netherlands	225
Greece	28	Japan	296

*1984 Statistics from World Bank Policy Study.

1.5 SCOPE OF HIGHWAY ENGINEERING

The road pavements are generally constructed on small embankments, slightly above the general ground level wherever possible, in order to avoid the difficult drainage and maintenance problems. The term road or roadway thus constructed is therefore termed 'highway' and the science and technology dealing with Road Engineering is generally called 'Highway Engineering'.

In the foregoing paragraphs, the need and the status of road transportation have been discussed. It is therefore logical to discuss the science of highway engineering which answers the questions as how highways are planned and designed and how are they constructed and maintained. Answers to each of these questions contribute to the development of the subject as outlined in Fig. 1.1.

In nutshell, it may be said that the highway engineering deals with various phase like development, planning, alignment, highway geometric design and location, highway traffic operation and its control, materials, pavement design, construction and maintenance, economic considerations, finance and administration.

Many other special aspect which are not included in the general layout given in Fig. 1.1 are problems related with development and construction of hill-roads roadside development including landscaping road architecture and arboriculture.

Scope of Highway Engineering

PHASES	DETAILS
Development, Planning and Locations	Historical background; Basis for Planning; Master Plan; Engineering surveys and highway alignment.
Highway Design, Geometrics and Structures	Road Geometrics and their Design; Rigid and Flexible Pavements; Design factors and thickness design; Overlay design; Design of drainage system.
Traffic Performance and its Control	Traffic studies analysis; Need for new road links; Traffic regulation and control; Intersection design and their controls with signs, signals, islands and markings.
Materials, Construction and Maintenance	Highway Materials and mix design; Highway construction; Earth work, construction of different types of pavements; Earthen, soi stabilized, water bound macadam, Bituminous surfaces an cement concrete roads; pavement failures; pavement evaluation Maintenance of pavements and drainage system.
Economics, Finance and Administration	Road user cost and economic analysis of highway projects; pavement types and maintenance measures; Highway Finance and phasing of expenditures; Administration.

Fig. 1.1 Scope of Highway Engineering

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PROBLEMS

1. Discuss the role of transportation in the economic and social activities of the country.
2. What are the different modes of transportation ? Explain the specific functions of each of them.
3. Compare the characteristic feature of different modes of transportation.
4. What are the characteristics of road transport in comparison with other systems ?
5. What, in your opinion were the chief causes of neglected conditions of road transportation in India ?
6. Explain the role of transportation in rural development in India ?
7. Outline and discuss the scope of the highway engineering study.

Chapter 2
Highway Development and Planning

2.1 HISTORICAL DEVELOPMENT OF ROAD CONSTRUCTION

2.1.1 Early Development

The oldest mode of travel obviously was on the foot-paths. Animals were also used to transport men and materials. Later simple animal drawn vehicles were developed and this became a common and popular mode of transportation for a very long period after the invention of *wheel*. This brought up the necessity of providing a *hard surface* for these wheeled vehicles to move on. Such a hard surface is believed to have existed in *Mesopotamia* in the period about 3500 B.C. The first road on which there is some authentic record is that of *Assyrian* empire constructed by about 1900 B.C. Only during the period of the *Roman* empire, roads were constructed in large scale and the earliest construction techniques known are of Roman Roads. The Romans constructed an extensive system of roads radiating in many directions from *Rome* through the empire mainly for military operations. Hence Romans are considered to be the pioneers in road construction.

2.1.2 Roman Roads

Many of the early Roman roads were of elaborate construction. Some of these roads are still in existence after over 2000 years. During this period of Roman civilization many roads were built of stone blocks of considerable thickness. The *Appian way* was built in 312 B.C. extending over 580 km which illustrate the road building technique used by Romans.

The main features of the Romans roads are :

(i) They were built straight regardless of gradients.

(ii) They were built after the soft soil was removed and a hard stratum was reached.

(iii) The total thickness of the construction was as high as 0.75 to 1.2 metres at some places, even though the magnitude of wheel loads of animal drawn vehicles was very low.

A typical cross section of Roman road is shown in Fig. 2.1. The construction procedure was as follows :

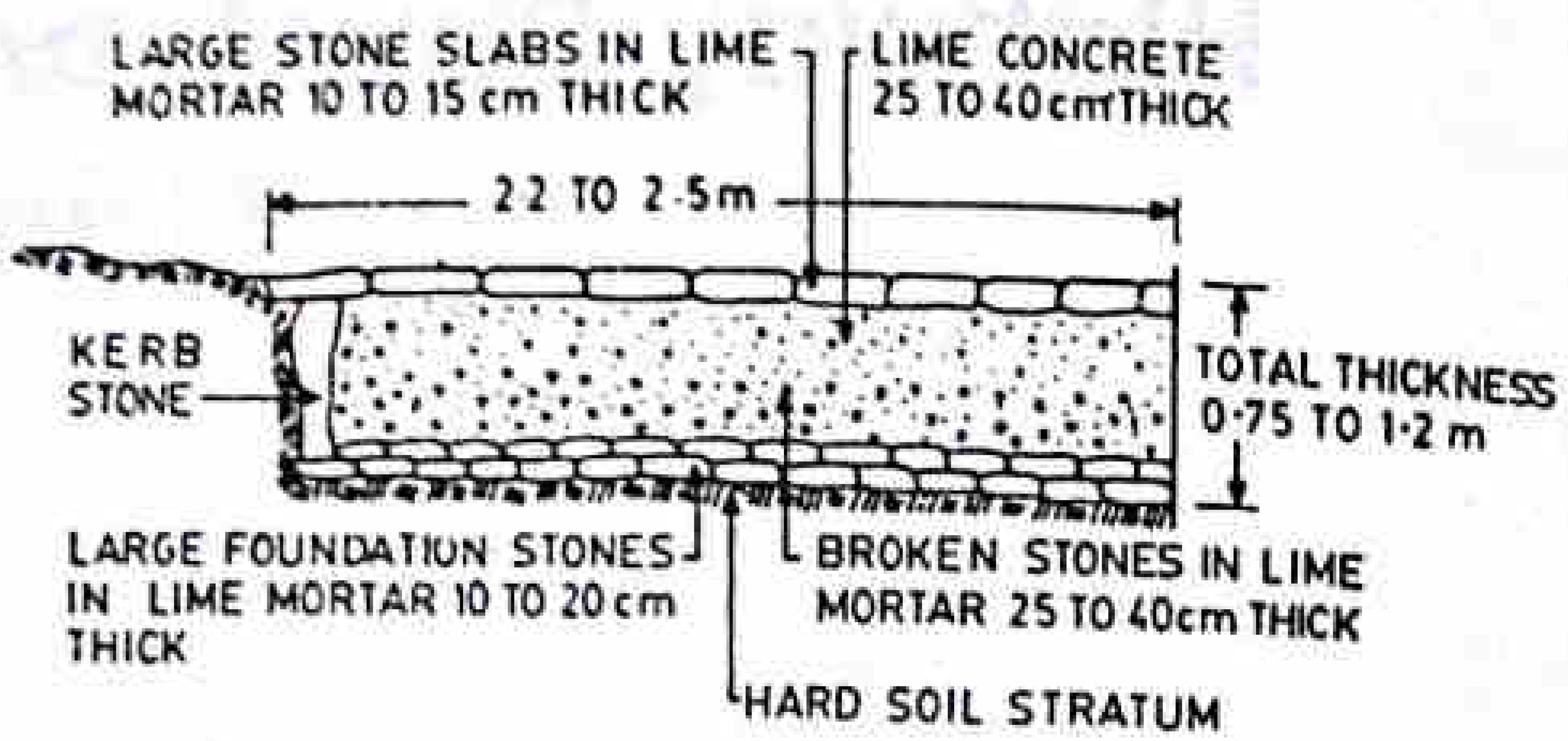


Fig. 2.1 Typical Cross section of Roman Road

(i) A trench of width equal to that of the carriage way was dug along a straight path by removing the loose soil from the top. The trench was cut upto a depth until a hard stratum was reached.

(ii) One or two layers of large foundation stones were laid in lime mortar at the bottom. The thickness of this bottom layer ranged from 10 to 20 cm. Vertical kerb stone were placed along the edges of the pavement.

(iii) A second layer of lime concrete with large size broken stones mixed in lime mortar was laid over the bottom course up to a thickness of 25 to 40 cm.

(iv) Another layer of lime concrete was laid over this with smaller broken stones mixed in lime mortar to a thickness of 25 to 40 cm, or even more if necessary.

(v) The wearing course consisting of dressed large stone blocks set in lime mortar was provided at the top. The thickness of these blocks also varied from 10 to 15 cm.

Obviously the above construction should have been much stronger than what was required for the animal drawn carts in those days. The enormous cost of construction cannot be justified at all, if this technique is compared with the modern trend of pavement design based on more scientific approaches.

2.1.3 Tresaguet Construction

After the fall of the Roman empire, their technique of road construction did not gain popularity in other countries. Until the eighteenth century there is no evidence of any new road construction method, except the older concept of using *thick* construction of road beads as the Roman did.

Pierre Tresaguet (1716-1796) developed an improved method of construction in France by the year 1964 A.D. Tresaguet developed several methods of construction which were considered to be quite meritorious. The main feature of his proposal was that the thickness of construction need be only in the order of 30 cm. Further due consideration was given by him to subgrade moisture condition and drainage of surface water. Tresaguet was the Inspector General of Roads in *France* from 1775 to 1785 and

so his method of construction was implemented in that country in 1775. During the regime of *Napoleon* the major development of road system in France took place.

The typical cross section of Tresaguet's road construction is given in Fig. 2.2 and the construction steps may be enumerated as below :

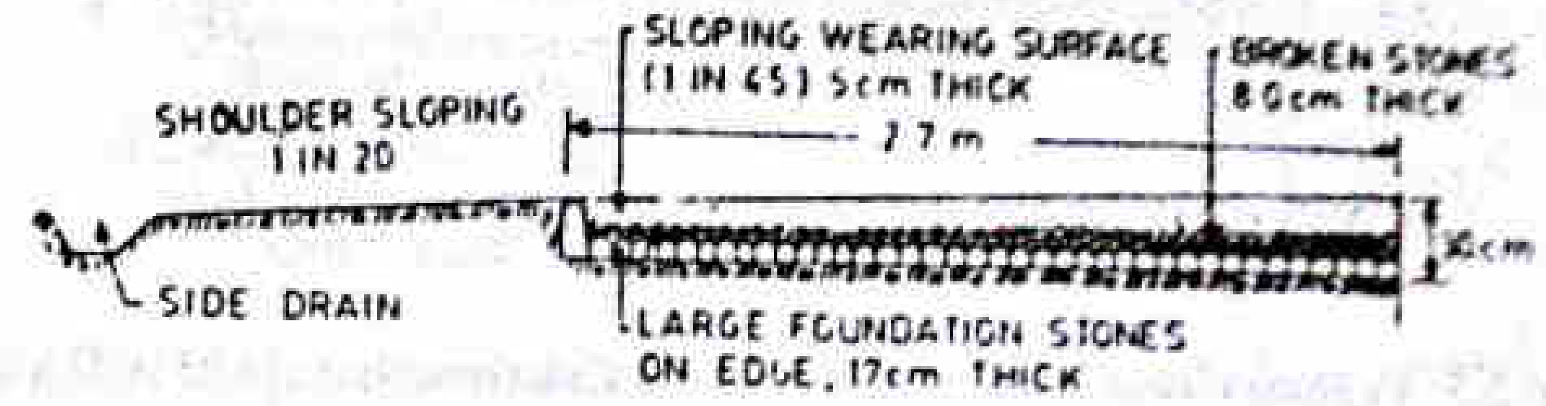


Fig. 2.2 Typical Cross Section of Tresaguet's Construction (1775 A.D.)

(i) The subgrade was prepared and a layer of large foundation stones were laid on edge by hand. At the two edges of the pavement large stones were embedded edgewise to serve as submerged kerb stones.

(ii) The corners of these heavy foundation stones were hammered and then the interstices filled with smaller stones. Broken stones were packed to a thickness of about 8 cm and compacted.

(iii) The top wearing course was made of smaller stones and compacted to a thickness of about 5 cm at the edges and gradually increased towards the centre, giving a cross slope of 1 in 45 to the surface, to provide surface drainage.

(iv) The shoulders were also provided cross slope to drain the surface water to the side drain.

2.1.4 Metcalf Construction

John Metcalf (1717-1810) was engaged on road construction works in *England* during the period when Tresaguet was working in France. He apparently followed the recommendations of *Robert Phillips* whose paper was presented in Royal Society. Metcalf was responsible for the construction of about 290 km of road in the northern region of England. As Metcalf was blind, much of his work was not recorded.

2.1.5 Telford Construction

Thomas Telford (1757-1834) began his work in early 19th century. He was the founder of the Institution of Civil Engineers at London. He also believed in using heavy foundation stones above the soil subgrade in order to keep the road foundation firm. He insisted on providing a definite cross slope for top surface of the pavement by varying the thickness of foundation stones.

A typical cross section of Telford's construction by the year 1803 is shown in Fig. 2.3.

The construction steps are given below :

(i) A level subgrade was prepared to designed width of about 9 meters.

(ii) Large foundation stones of thickness 17 to 22 cm were laid with hand with their largest face down so as to be laid in a stable position. The stones of lesser thickness

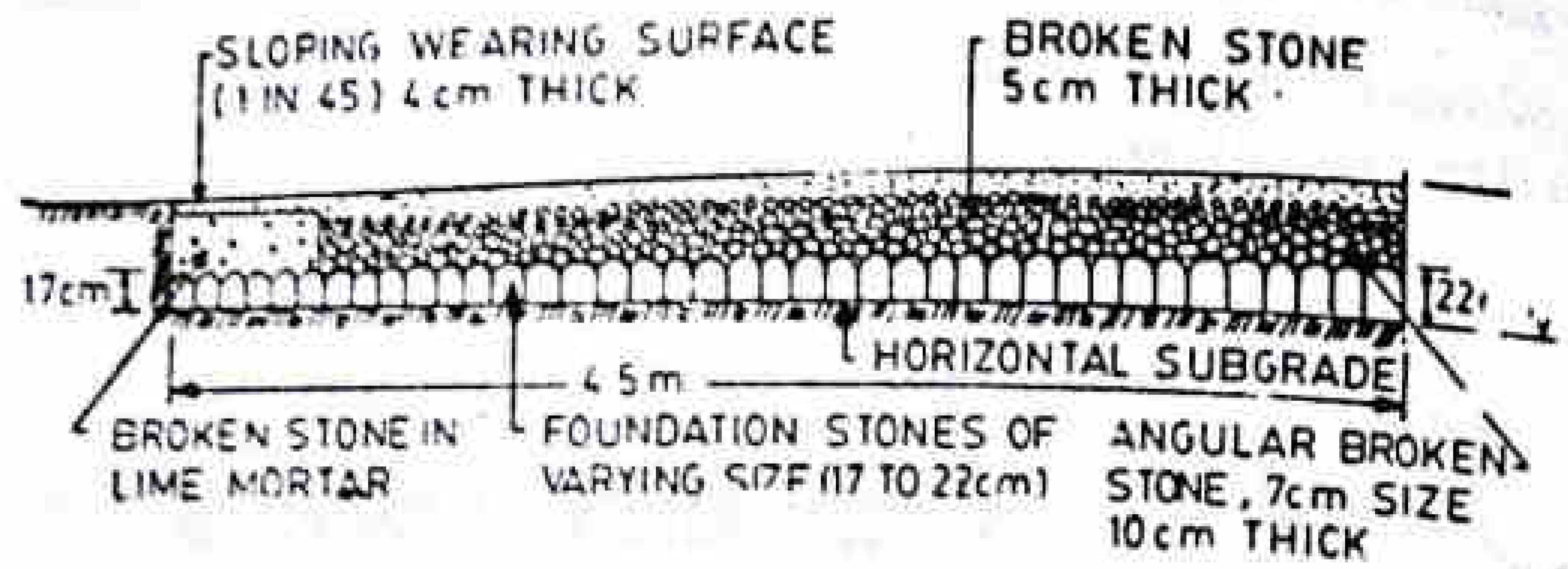


Fig. 2.3 Typical Cross Section of Telford's Construction (1803 A.D.)

(17 cm) were placed towards the edges and stones of increasing thickness were laid towards the centre. At the centre the largest stones of approximate thickness 22 cm were used such that these foundation stones of varying thickness provide the cross slope designed by Telford.

(iii) The interstices between foundation stones were filled with smaller stone and chipping and properly beaten down.

(iv) The central portion of about 5.5 metre width was covered with two layers of angular-broken stones to compacted thickness of 10 and 5 cm. These layers were initially rammed and later allowed to be compacted under the traffic and get consolidated by the rains.

(v) A certain width of the pavement towards the edges was constructed by compacted broken stones, 15 cm thick, sometimes in lime mortar instead of using the kerb stones so as to provide lateral stability.

(vi) A binding layer of wearing course 4 cm thick was constructed on the top using gravel. The finished surface had a cross slope of about 1 in 45.

Telford proposed to provide cross drains at intervals of about 90 meters. They were usually laid below the foundation level as the interstices were large enough to allow the water to percolate from top to the bottom of the construction and thus soften the level subgrade.

2.1.6 Macadam Construction

John Macadam (1756-1836) put forward an entirely new method of road construction as compared to all the previous methods. The first attempt to improve the road condition was made by him in 1815. Macadam was the Surveyor General of Roads in *England* and his new concept of road construction became known by the year 1827.

A typical cross section of Macadam's construction is shown in Fig. 2.4.

The most important modifications made in Macadam's methods with respect to the older methods are :

(i) The importance of subgrade drainage and compaction were recognised and so the subgrade was compacted and was prepared with a cross slope of 1 in 36.

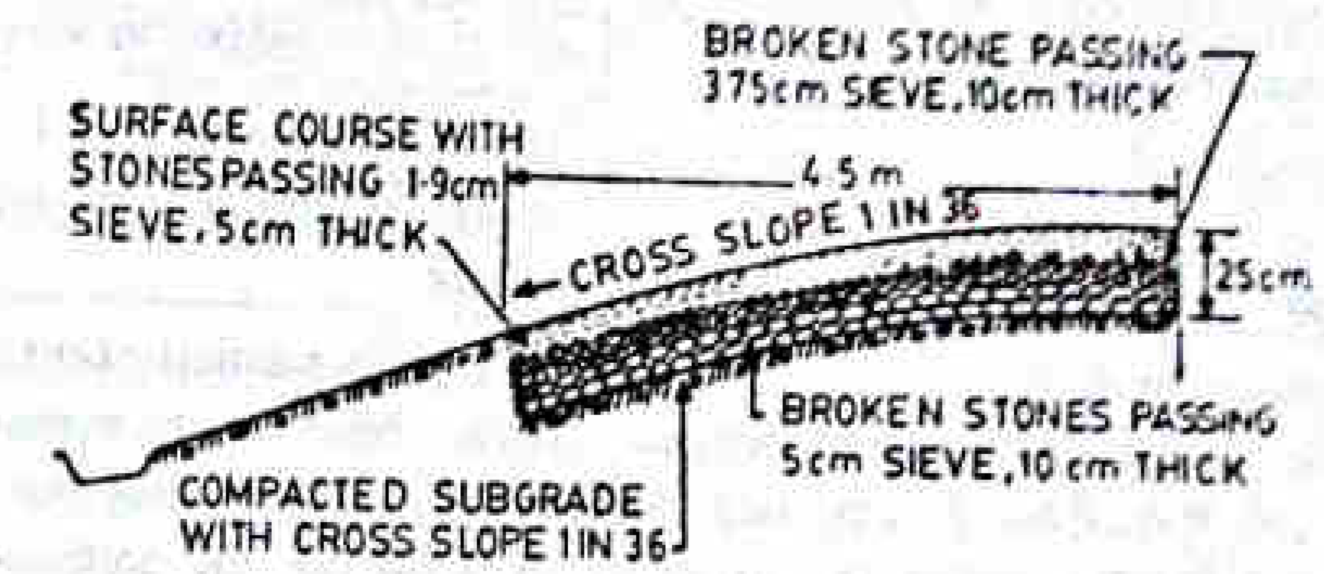


Fig. 2.4 Typical Cross Section of Macadam's Construction (1827 A.D.)

(ii) Macadam was the first person to suggest that heavy foundation stones are not at all necessary to be placed at the bottom layer of construction. He realised that the subgrade being the lowest portion of the pavement should be prepared properly and kept drained so as to carry the load transmitted through the pavement. Compacted layer of smaller size broken stones placed at the bottom, according to Macadam, could replace, with advantage, the heavy foundation stones.

(iii) Though the total thickness of construction, was less than previous methods, this technique could serve the purpose in a better way, due to better load dispersion characteristics of compacted broken stone aggregates of smaller sizes.

(iv) The size of broken stones for the top layer was decided based on the stability under animal drawn vehicles. The pavement surface was also prepared with a cross slope of 1 in 36 for drainage of surface water.

Macadam's method is the first method based on scientific thinking. It was realised that the stresses due to wheel load of traffic gets decreased at the lower layers of the pavement and therefore it is not necessary to provide large and strong boulder stones as foundation or soling course at the lowest layer of the pavement. This method became very popular far and wide. Various subsequent improved methods were based on Macadam's construction and some of the methods still in use are known after his name, such as water bound macadam, penetration macadam and bituminous macadam constructions.

The construction steps are :

(i) Subgrade is compacted and prepared with a cross slope of 1 in 36 upto a desired width (about 9 metres).

(ii) Broken stones of a strong variety, all passing through 5 cm size sieve were compacted to a uniform thickness of 10 cm.

(iii) The second layer of strong broken stones of size 3.75 cm was compacted to thickness of 10 cm.

(iv) The top layer consisted of stones of size less than 2 cm compacted to a thickness of about 5 cm and finished so that the cross slope of pavement surface was also 1 in 36.

The Macadam and Telford methods of construction differ considerably though both the methods were put forward in the early nineteenth century.

The two methods have been compared here :

Macadam method	Telford method
(i) The subgrade was given a cross slope of 1 in 36 to facilitate subgrade drainage.	The subgrade was kept horizontal and hence subgrade drainage was not proper.
(ii) The bottom layer of pavement or the sub-base course consisted of broken stones of less than 5 cm size to uniform thickness equal to 10 cm only.	Heavy foundation stones of varying sizes, about 17 cm towards the edges and 22 cm towards the centre were hand packed and prepared to serve as sub-base course.
(iii) Base and surface courses consisted of broken stones of smaller sizes to compacted thickness of 10 and 5 cm respectively and the top surface was given a cross slope of 1 in 36.	Two layers of broken stones were compacted over the foundation stones before laying the wearing course, 4 cm thick with a cross slope of 1 in 45.
(iv) The total thickness of pavement construction was kept uniform from edge to centre to a minimum value of only 25 cm.	The total thickness of construction varied from about 35 cm at the edge to about 41 cm at the centre.

2.1.7 Further Developments

Macadam's method of construction gained recognition as a scientific method of construction and hence was adopted by various countries with slight modifications. One of the most popular methods which is even now prevalent in many countries is the *water bound macadam* (WBM) construction, known after Macadam's technique. In this method the broken stones of the base course and surface course, if any, are bound by the stone dust in presence of moisture and hence the name. WBM roads are in use in India both as a finished pavement surface for minor roads and as a good base course for superior pavements carrying heavy traffic. There are also bituminous construction methods which are known after Macadam. The methods adopted in our country include the bitumen bound macadam and penetration macadam. The details of the construction methods have been given in the Chapter, 'Highway Construction'.

The water bound macadam roads were considered to be one of the superior methods of construction until the fast moving vehicles started using these roads. Dust is formed on the road surface during dry weather due to the crushing and abrading action of steel-tyred animal drawn vehicles. This dust is easily raised by the fast moving automobiles. Further during monsoons, mud is formed and is churned again due to the movement of automobiles. Under the combined action of the mixed traffic and under adverse weather conditions the WBM roads could not last long. In order to minimise the dust nuisance, several dust palliatives including heavy oils and bituminous materials were tried with varying degrees of success.

The next development was the penetration and bituminous macadam roads and other types of surface dressing methods using bituminous materials. For better performance superior bituminous mixes like the bituminous carpet and bituminous concrete were also developed in a scientific way.

The use of cement concrete for roads has been popular even prior to the use of bituminous mixes. The cement concrete roads could be designed to kept up the heaviest loads expected on the roads even in adverse soil and climatic conditions and to last for a long service life. They are known to give a good and even riding surface. Due to the high initial cost involved in the construction of cement concrete road, it is not being extensively used in our country at present.

As the main problem in developing countries like India is to have maximum road length at minimum cost, the only solution is to resort to the construction of low cost roads and stage construction of roads. Hence the best utilisation of locally available and the cheapest materials have to be made in road construction. In this respect there is good scope for the use of *soil stabilization* and other low cost pavement materials. There are several techniques of soil stabilization which have been explained in a later chapter of the book. The choice of the method of stabilization depends on several factors such as the soil type, availability of stabilizers, climatic conditions, the component of pavement which is being constructed and the traffic.

2.2 HIGHWAY DEVELOPMENT IN INDIA

2.2.1 Roads in Ancient India

The excavations of Mohenjo-daro and Harappa have revealed the existence of roads in India as early as 25 to 35 centuries B.C. Old records reveal that in early periods the roads were considered indispensable for administrative and military purposes. The ancient scriptures refer to the existence of roads during the Aryan period in the fourth century B.C. *Kautilya* the first prime minister of Emperor *Chandra Gupta Maurya*, laid down the rules in the literary piece titled *'Arthasastra'*. Rules have been mentioned about regulating the depth of roads for various purposes and for different kinds of traffic. Mention has been made regarding the punishment for obstructing roads. In the beginning of fifth century A.D. emperor *Ashoka* had improved the roads and the facilities for the travellers.

2.2.2 Roads in Mughal Period

During the Pathan and Mughal periods, the roads of India were greatly improved. Some of the highways either built or maintained by Mughals received great appreciation from the foreign visitors who visited India during that periods. Roads were built running from North-West to the Eastern areas through the Gangetic plains, linking also the coastal and central parts.

2.2.3 Roads in Nineteenth Century

At the beginning of British rule, the conditions of roads deteriorated. The economic and political shifts caused damage to a great extent in the maintenance of the road transportation. The fall of Mughal empire led therefore to the scant attention to the communication. Prior to the introduction of railways, a number of trunk roads were metalled and bridges were provided. this was mainly done on the remains of old roads which existed, under the supervision of the British Military Engineers. In fact these roads connected important military and business centres.

Military maintenance was not quite adequate and in 1865 *Lord Dalhousie*, when he was Governor-General formed the *Public Works Department* in more or less the same form that exists today. The construction of the *Grand Trunk Road* was undertaken by this new department. Immediately with the development of railways, attention of the Government was shifted from road development except for providing feeder roads and the railway was gaining the privileges.

2.2.4 Jayakar Committee and the Recommendations

After the first World War, motor vehicles using the roads increased and this demanded a better road network which can carry both bullock cart traffic and motor vehicles. The existing roads when not capable to withstand the *mixed traffic* conditions. A resolution was passed by both Chambers of the Indian Legislature 1927 for the appointment of a committee to examine and report on the question of road development in India. In response to the resolution, Indian Road Development committee was appointed by the government with *M.R. Jayakar* as Chairman, in 1927.

The Jayakar Committee submitted its report by the year 1928. The most important recommendations made by the committee are :

(i) The road development in the country should be considered as a national interest as this has become beyond the capacity of provincial governments and local bodies.

(ii) An extra tax should be levied on petrol from the road users to develop a road development fund called *Central Road Fund*.

(iii) A semi-official technical body should be formed to pool technical know how from various parts of the country and to act as an advisory body on various aspects of roads.

(iv) A research organisation should be instituted to carry out research and development work and to be available for consultations.

Most of the recommendations of the Jayakar Committee were accepted by the government, and the major items were implemented subsequently. The Central Road Fund was formed by the year 1929, the semi-official technical body called the Indian Roads Congress was formed in 1934 and the Central Road Research Institute was started in 1950.

Central Road Fund

Based on the authority of a resolution adopted by the Indian Legislature, the Central Road Fund (C.R.F.) was formed on 1st march 1929. The consumers of petrol were charged an extra leavy of 2.64 paisa per litre (then two annas per gallon) of petrol to build up this road development fund 20 percent of the annual revenue is to be retained as a *Central Reserve*, from which grants are to be given by the Central Government for meeting expenses on the administration of the road fund, road experiments and research on road and bridge projects of special importance. The balance 80 percent is to be allotted by the Central Government to the various states based on actual petrol consumption or revenue collected. The accounts of the Central Road Fund are maintained by the Accountant General of Central Revenues and the control on the expenditure is exercised by the Roads Wing of Ministry of Transport. Recently the rate of collection of the levy towards the CRF has been revised in order to augment the revenue under this fund.

Indian Roads Congress

At the instance of central government a semi-official technical body known as *Indian Roads Congress* (IRC) was formed in 1934. This, it may be recalled, is one of the main recommendations made by the Jayakar Committee. The Indian Roads Congress was constituted to provide a forum for regular pooling of experience and ideas on all matters affecting the planning, construction and maintenance of roads in India, to recommend standard specifications and to provide a platform for the expression of professional opinion on matters relating to road engineering including such questions as those of organisation and administration. The IRC has played important role in the formulation of the three 20-year road development plans in India. Now the Indian Roads Congress has become an active body of national importance controlling specifications, standardisation and recommendations on materials, design and construction of roads and bridges. The IRC publishes journals, research publications, standards specifications guidelines and other special publications on various aspect of Highway Engineering. The technical activities of the IRC are mainly carried out by the Highway research Board and several committees and subcommittees consisting of experts in each subject. The IRC works in close collaboration with Roads Wing of the Ministry of Surface Transport, Government of India.

Economic depression during the thirties of this century delayed the road development programmes. During this period the share from the Central Road Fund was almost the only source for highway financing. During the second world war intensive efforts were made to develop the road net work essential required for strategic considerations. These projects were substantially supported by defence services funds.

After the second world war, there was a revolution in respect of automobiles using the roads in our country and a large number of military vehicles started plying on the roads. Thus the road development could not cope up with rapid increase in road vehicles and so the existing roads started deteriorating fast. The need for proper highway planning was urgently felt by this time by the authorities.

Motor Vehicle Act

In 1939 the Motor Vehicles Act was brought into effect by Government of India to regulate the road traffic in the form of traffic laws, ordinances and regulations. The three phases primarily covered are control of the driver, vehicle ownership and vehicle operation on roads and in traffic stream. The Motor Vehicle Act has been appended with several ordinances subsequently. The Motor Vehicles Act has been revised in the year 1988.

2.2.5 Nagpur Road Conference

A conference of the Chief Engineers of all the states and provinces was convened in 1943 by the Government of India at Nagpur, at initiative of the Indian Roads Congress to finalise the first road development plan for the country as a whole. This is a landmark in the history of road development in India, as it was the first attempt to prepare a co-ordinated road development programme in a planned manner. In this first 20 year road development plan, popularly known as the *Nagpur Road Plan*, all roads were classified into five categories and a twenty year development programme for the period 1943-63 was finalised. At the end of this plan the target road length aimed at was 16 km per 100 square km area of the country, During the first and second five-year plan periods (1951-56 and 1956-61), the road development programme also was systematic and hence the Nagpur plan target of total road length was achieved about two years ahead, in 1961. But even as early as 1957 a meeting of the Chief Engineers of the central and state Governments was convened, to consider the future road development programme. A committee was appointed to prepare the Second Twenty-year Road Development plan starting from the year 1961.

Central Road Research Institute

In the year 1950 the Central Road Research Institute (CRRI) was started at New Delhi for research in various aspect of highway engineering. It may be indicated that one of the recommendations of Jayakar Committee report was to set up a central organisation for research and dissemination of information.

The CRRI is one of the national laboratories of the Council of Scientific and Industrial Research; the institute is mainly engaged in applied research and offers technical advice to state governments and the industries on various problems concerning roads.

National Highway Act

In 1956 the National Highway Act was passed. The main features of the act are :

(i) the responsibility of development and maintenance of the national highway (NH) to be provisionally taken by the central government.

(ii) the central government to be empowered to declare any other highway as NH or to omit any of the existing national highways from the list.

2.2.6 Second Twenty Year Road Development Plan 1961-81

The second twenty year road development plan for the period 1961-81 was initiated by the IRC and was finalised in 1959 at the meeting of the Chief Engineers and the same was forwarded to the Central government. This road development plan is also known as Bombay Road Plan. The plan gave due consideration to the development that are taking place and developments that have to take place in our country in various fields during the plan period. The target road length at the end of this second 20 year plan was almost double that of the Nagpur road plan target i.e. a total road length of 10,57,330 km or about 32 km per hundred sq. km area. An outlay of Rs. 5,200 crores for the period ending 1980-81 was envisaged for this second twenty year plan, based on 1958 price level. The construction of 1,600 km of Express ways was also then included in the plan.

During the third five year plan period 1961-66, the annual plans 1966-69 and the fourth five year plan period 1969-74, the road development in India continued at a steady pace. The total length of all categories of roads achieved by the year 1974 was 11.45 lakhs km, the density of road length being 34.8 km per 100 sq. km area, which is higher than the 1981 target of the Second Twenty Year Road Development Plan1961-81. Thus there was an immediate need to prepare the third long term road development plan for the country by then. However, due to change in planning policies during the fifth plan period 1974-78, annual plans 1978-80 and the sixth five year 1980-85, the preparation of the third long term road development plan got delayed.

The Third Twenty Year Road Development Plan for the period 1981-2001 was prepared by a committee of experts and was approved by the Council of the Indian Roads Congress and also at the meeting of the Chief Engineers of the Country in the year 1984.

Highway Research Board

The Highway Research Board of the Indian Roads Congress was set up in 1973 with a view to give proper direction and guidance to road research activities in India. The board is expected to act as a national body for co-ordination and promotion of highway research. The Highway Research Board (HRB) has recommended suitable financial allocation of research by central and state governments and has chosen high priority research schemes for being taken up first.

The objective of IRC Highway Research Board are :

(i) To ascertain the nature and extent of research required.

(ii) To correlate research information from various organisations in India and abroad with a view to exchange publications and information on roads.

(iii) To co-ordinate and conduct correlation services.

(iv) To collect and disseminate results on research

(v) To channelise consultative services

There are three technical committees of the HRB for (i) identification, monitoring and research application (ii) road research evaluation and dissemination and (iii) bridge research, evaluation and dissemination.

2.2.7 National Transport Policy Committee

The Government of India appointed the National Transport Policy Committee (NTPC) in the year 1978 to prepare a comprehensive national transport policy for the country for the next decade or so, keeping in view the objectives and priorities set out in the five year plans. The NTPC report was made available in the year 1980 and many of the major recommendations of this report have been accepted by the Government of India. Some of the important recommendations of the NTPC report relate to the liberalisation of the transport sector, inclusion of transport in the priority sector, optimal in-er-modal mix between railway and road transport based on resource-cost consideration and energy conservation. A number of suggestions were made on the road development, these include the need to take into account the requirements of roads in rural, hilly and tribal areas in the next perspective road development plans, strengthening of National Highway system, increase in funds for the maintenance of roads and to connect all the villages with all-weather low-cost roads within next twenty years. Separate recommendations were also made for various factors connected with the development and growth of road transport by the year 2001.

2.2.8 Third Twenty Year Road Development Plan 1981-2001

The Third Twenty Year Road Development plan 1981-2001 was prepared by the Road Wing of the Ministry of Shipping and Transport with the active co-operation from a number of organisations and experts in the field of Highway Engineering and Transportation. This document was released during the 45th Annual Session and the Golden Jubilee celebrations of the Indian Road Congress in February 1985 at Lucknow. Therefore this road development, plan for 1981-2001 is also called 'Lucknow Road Plan'. This plan has been prepared keeping in view the growth pattern envisaged in various fields by the turn of the century. Some of the points which were given due consideration while formulating the plan are improvement of transportation facilities in villages, towns and small cities, conservation of energy, preservation of environmental quality and improvement in road safety.

This twenty year road plan aims at increasing the total road length (including urban and project roads) from 15,02,700 km in the year 1981 to 27,02,000 km by the year 2001. This will result in an increase in road density from 46 km per 100 sq. km in the year 1981 to 82 km per 100 sq. km by the year 2001. As the development of National Highways fell short of the targets set by the first two twenty year road development plans, the present road plan of 1981-2001 has set the target length of NH to be completed by the end of seventh, eighth and ninth five year plan periods.

2.2.9 Review of Highway Development in India after Independence

When India achieved independence on 15th August 1947, the total road length in the country was 3,88,226 km with the density of road length working out to about 11.8 km per 100 sq. km area, out of this the length of surfaced roads was only about 36.8 percent. Since then the pace of road development in the country has improved considerably. The total road length in the country increased from 3,97,600 km in the year 1950-51 to 15,02,700 km by the year 1980-81.

There has also been a tremendous pressure on road transportation due the rapid growth in the number of road vehicles and considerable increase in freight and passenger traffic carried by roads during the above thirty year period. The total number of motor vehicles in the country increased from about three lakhs to over 44 lakhs during the period 1951-1981. The freight traffic on roads increased from about 5.5 to over 104 billion (i.e. 1×10^9) tonne-km and the passenger traffic from about 23 to over 315 billion passenger-km during the above thirty years. Obviously the above growth in the road traffic has been due to overall growth in the National income and the population and due to the increase in agricultural and industrial production. The annual revenue from taxation on road transport sector also increased from about Rs. 61 crores to Rs. 2,388 crores during the above thirty year period.

The growth of total road length, density of road length and the length of surfaced roads in the country during the period since independence are given in Table 2.1. The total revenue from road transport due to taxation and the total expenditure on roads including development and maintenance expenditure on roads by the centre and the states are also given in this table. It may be seen that only a fraction of the revenue from road transport is being spent on development and maintenance of roads.

The development of National Highways in India has not been at the desired pace. Though the achievements in terms of total road length by the end of Nagpur Road Plan in 1961 and Bombay Road Plan in 1981 exceeded the targets of the two twenty-year road development plans, the length of NH achieved fell short of both the plan targets. As the NH system caters for a high proportion of traffic in the country, provision has now been made for the improvement of deficiencies in the existing NH system and also for the construction of additional length of NH to cater for the growing traffic needs upto the end of this century.

Table 2.1 Growth of Road Length, Revenue and Expenditure on Roads in the Country

Year	Total road length in lakh km	Density of total road length in km per 100 sq. km area	Length of surfaced roads in lakh km	Revenue from road transport taxation in crores	Expenditure on roads in crores Rs.
1947	3.88	11.8	1.43		-
1950-51	3.98	12.1	1.56	61.0	-
1960-61	7.05	21.4	2.34	166.9	159.8
1965-66	8.80	26.8	3.43	399.1	179.7
1970-71	10.12	30.8	4.36	683.2	257.6
1975-76	12.49	38.0	5.51	1436.6	502.6
1977-78	13.72	41.7	5.96	1587.6	693.5
1980-81	15.03	45.7	6.92	2387.6	1209.3

2.3 NECESSITY OF HIGHWAY PLANNING

In the present era planning is considered as a pre-requisite before attempting any development programme. This is particularly true for any engineering work, as planning is the basic requirement for any new project or an expansion programme. Thus highway planning is also a basic need for highway development. Particularly planning is of great importance when the funds available are limited whereas the total requirement is much higher. This is actually the problem in all developing countries like India as the best utilisation of available funds has to be made in a systematic and planned way.

The objects of highway planning are briefly given below :

(i) To plan a road net work for efficient and safe traffic operation, but at minimum cost. Here the costs of construction, maintenance and renewal of pavement layers and the vehicle operation costs are to be given due consideration.

(ii) To arrive at the road system and the lengths of different categories of roads which could provide maximum utility and could be constructed within the available resources during the plan period under consideration.

(iii) To fix up date wise priorities for development of each road link based on utility as the main criterion for phasing the road development programme.

(iv) To plan for future requirements and improvements of roads in view of anticipated developments.

(v) To work out financing system.

2.4 CLASSIFICATION OF ROADS

2.4.1 Types of Roads

The different types of roads are classified into two categories, depending on whether they can be used during different seasons of the year :

(i) All-weather roads and

(ii) Fair-weather roads. *All weather roads* are those which are negotiable during all weather, except at major river crossings where interruption to traffic is permissible upto a certain extent, the road pavement should be negotiable during all weathers. Roads which are called *fair weather roads*; on these roads, the traffic may be interrupted during monsoon season at causeways where streams may overflow across the road.

Based on the type of the carriage way or the road pavement, the roads are classified as :

(i) *paved roads*, if they are provided with a hard pavement course which should be atleast a water bound macadam (WBM) layer and

(ii) *unpaved roads*, if they are not provided with a hard pavement course of atleast a WBM layer. Thus earth roads and gravel roads may be called unpaved roads.

Based on the type of pavement surfacing provided; the road types are divided as :

(i) *surface roads*, which are provided with a bituminous or cement concrete surfacing and

(ii) *unsurfaced roads* which are not provided with bituminous or cement concrete surfacing. The roads provided with bituminous surfacing are also called *black toped roads*.

2.4.2 Methods of Classification of Roads

The roads are generally classified on the following basis :

(a) Traffic volume

(b) Load transported or tonnage

(c) Location and function

The classification based on traffic volume or tonnage have been arbitrarily fixed by different agencies and there may not be a common agreement regarding the limits for each of classification group. Based on the traffic volume, the roads are classified as heavy, medium and light traffic roads. These terms are relative and so the limits under each class should be clearly defined and expressed as vehicles per day etc. Likewise the classification based on load or tonnage is also relative and the roads may be classified as class I, II etc. or class A, B etc. and the limits may be expressed as tonnes per day.

The classification based on location and function should therefore be a more acceptable classification for a country as they may be defined-clearly. The Nagpur Road Plan classified the roads in India based on location and function into following five categories and described in section 2.4.3.

(i) National Highways (NH)

(ii) State Highways (SH)

(iii) Major District Roads (MDR)

(iv) Other District Roads (ODR) and

(v) Village Roads (VR)

2.4.3 Classification of Roads by Nagpur Road Plan

(i) *National Highways (NH)* are main highways running through the length and breadth of India, connecting major ports, foreign highways, capitals of large states and large industrial and tourist centres including roads required for strategic movements for the defence of India.

It was agreed that a first step *National Trails* should be constructed by the Centre and that latter these should be converted into roads to suit the traffic conditions. It was specified that national highways should be the frame on which the entire road communication should be based and that these highways may not necessarily be of same specification, but they must give an uninterrupted road communication through out India and should connect the entire road network.

All the national highways are assigned the respective numbers. The highway connecting *Delhi-Ambala-Amritsar* is denoted as NH-1, whereas a bifurcation of this highway beyond *Jalandar to Srinagar and Uri* is denoted NH-1-A. The highway connecting *Maduri and Rameswaram* is NH-49 and *Bombay-Agra* road is NH-3. A map showing National Highways is given in Plate *I*.

(ii) *State Highways* (SH) are arterial roads of a state, connecting up with the national highways of adjacent state, district head quarters and important cities within the state and serving as the main arteries for traffic to and from district roads.

These highways are considered as main arteries of commerce by roads within a state or a similar geographical unit. In some places they may even carry heavier traffic than some of the national highways but this will not alter their designation or function. The NH and SH have the same design speed and geometric design specifications.

(iii) *Major District Roads* (MDR) are important roads within a district serving areas of production and markets and connecting those with each other or with the main highways of a district. The MDR has lower speed and geometric design specifications than NH/SH.

(iv) *Other District Roads* (ODR) are roads serving rural areas of production and providing them with outlet to market centres, taluk head quarters, block development head quarters or other main roads. These are of lower design specifications than MDR.

(v) *Village Roads* (VR) are roads connecting villages or groups of villages with each other to the nearest road of a higher category.

It was specified that these village roads should be in essence farm tracks, but it was desired that the prevalent practice of leaving such tracks to develop and maintain by themselves should be replaced by a plan for a designed and regulated system.

A general note was assigned by the Nagpur Road Conference regarding the economics of road construction that all roads of whatever type or class, should be so constructed that maintenance and capital costs over a period of 20 years will be minimum. The responsibility of construction and maintenance of national highways was decided to be with the central government; it was stated that "Centre should select the national highways and trails, accord priorities and pay for all construction and maintenance".

2.4.4 Modified Classification of Road System by Third Road Development Plan, 1981-2001

The roads in the country are now classified into three classes, for the purpose of transport planning, functional identification, earmarking administrative jurisdictions and assigning priorities on a road net work viz.;

(i) Primary system

(ii) Secondary system and

(iii) Tertiary system or rural roads.

Primary system consists of two category

(a) Expressways and

(b) National Highways (NH)

Expressways are a separate class of highways with superior facilities and design standards and are meant as through routes having very high volume of traffic. The expressways are to be provided with divided carriage ways, controlled access, grade separations at cross roads and fencing. These highways should permit only fast moving vehicles. Expressways may be owned by the Central Government or a State Government, depending on whether the route is a National Highway or State Highway.

The Secondary system consists of two categories of roads

(a) State Highways (SH) and

(b) Major District Roads (MDR)

The Tertiary system are rural roads and these consists of two categories of roads :

(a) Other District Road (ODR)

(b) Village Roads (VR)

The definitions of NH, SH, MDR, ODR and VR are the same as given under classification of roads by Nagpur Road Plan in Art. 2.4.3.

2.4.5 Classification of Urban Roads

The road system within urban areas are classified as Urban Roads and will form a separate category of roads to be taken care by the respective urban authorities. The length of urban roads are not included in the targets of the Third Twenty Year Road Development Plan 1981-2001.

The urban roads, other than expressways, are classified as :

(i) Arterial roads

(ii) Sub-arterial roads

(iii) Collector streets and

(v) Local streets

Arterials and sub-arterials are streets primarily for through traffic on a continuous route, but the sub-arterials have a lower level of traffic mobility than the arterials. Collector streets provide access to arterial streets and they collect and distribute traffic from and to local streets which provide access to abutting property.

2.5 ROAD PATTERNS

The various road patterns may be classified as follows :

(a) Rectangular or block pattern

(b) Radial or star and block pattern

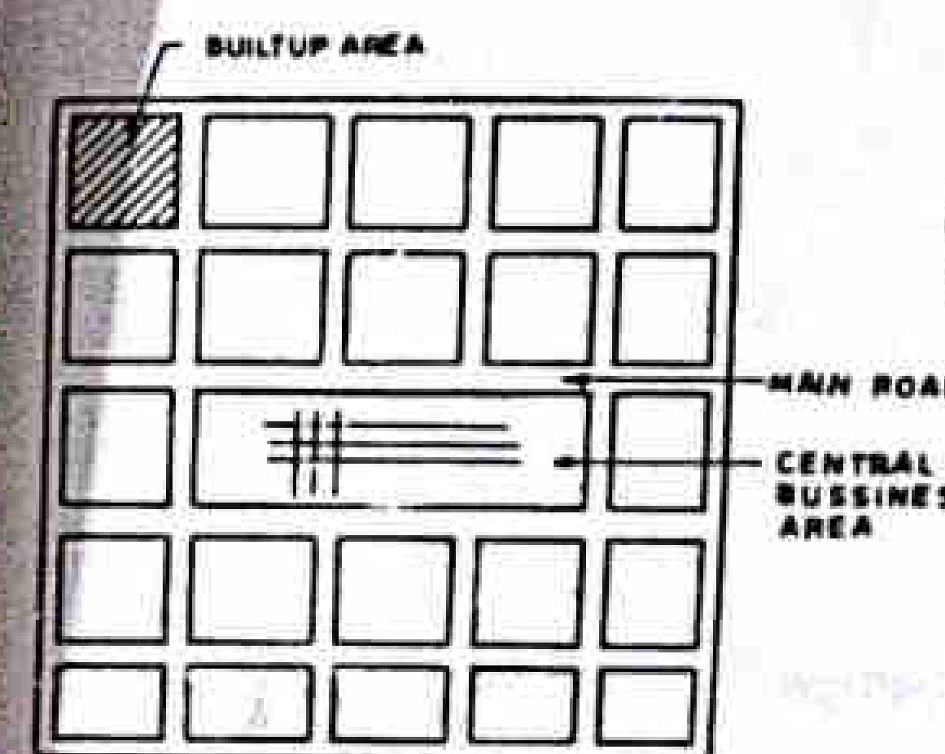
(c) Radial or star and circular pattern

(d) Radial or star and grid pattern

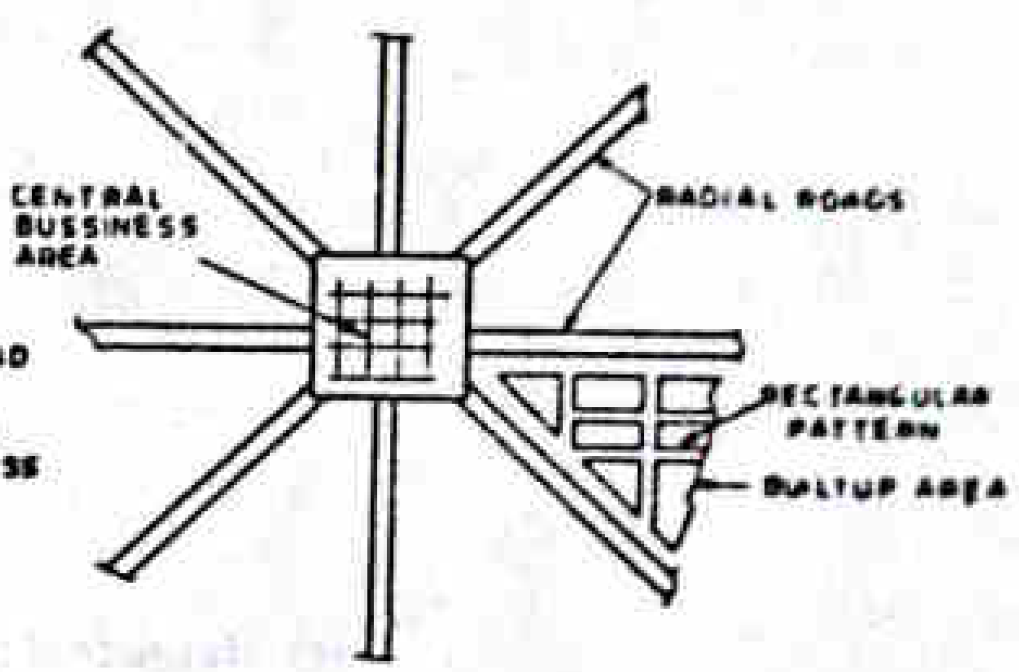
(e) Hexagonal pattern

(f) Minimum travel pattern

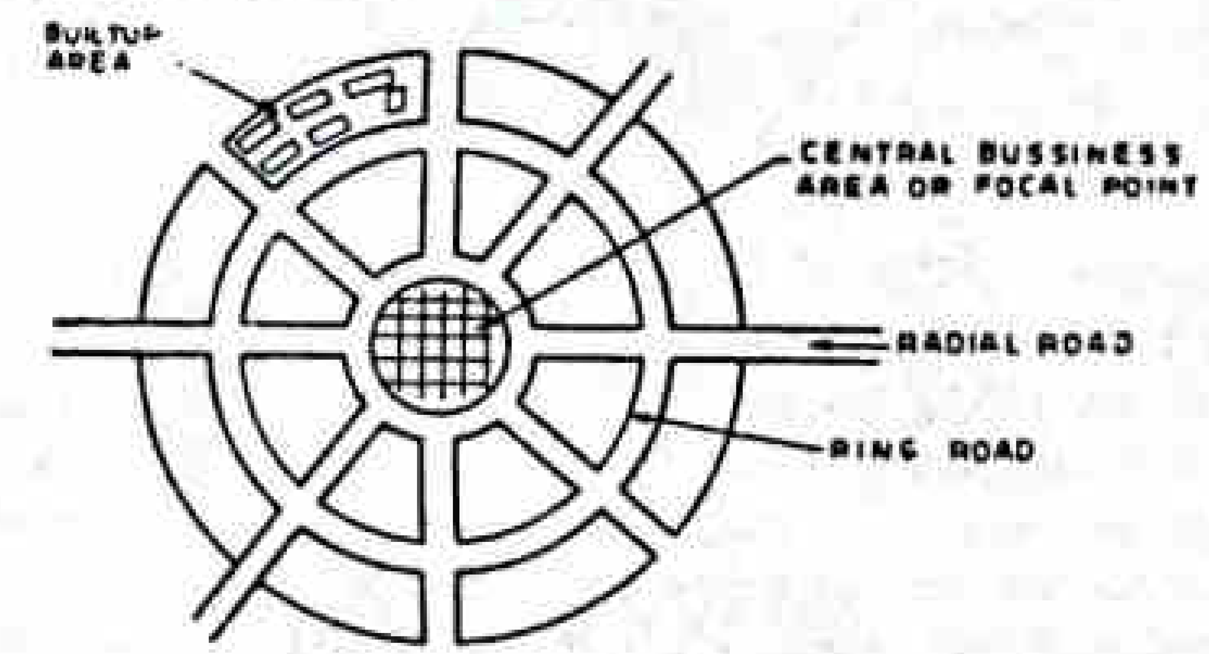
These have been shown in Fig. 2.5 a, b, c, d, e & f.



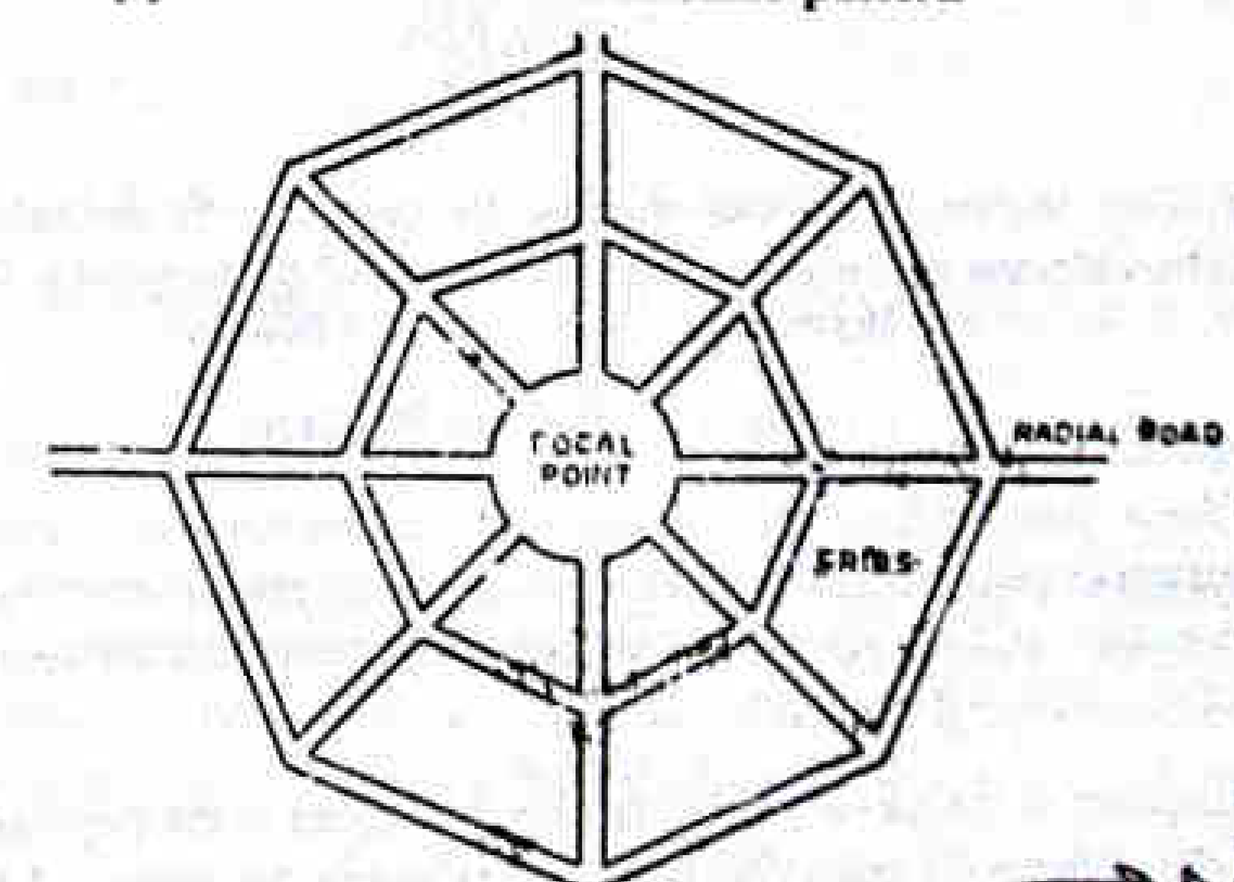
(a) Rectangular or block pattern



(b) Radial or star and block pattern

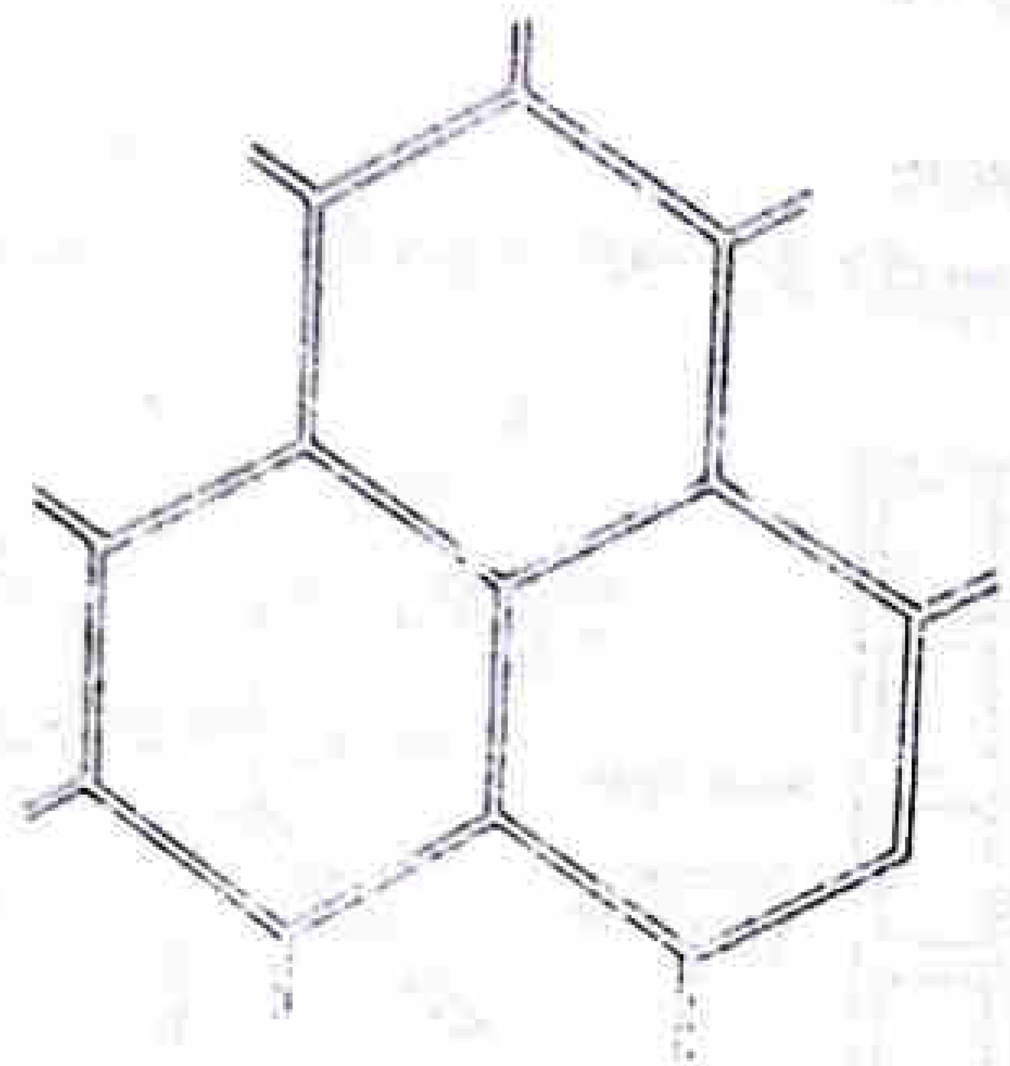


(c) Radial or star and circular pattern

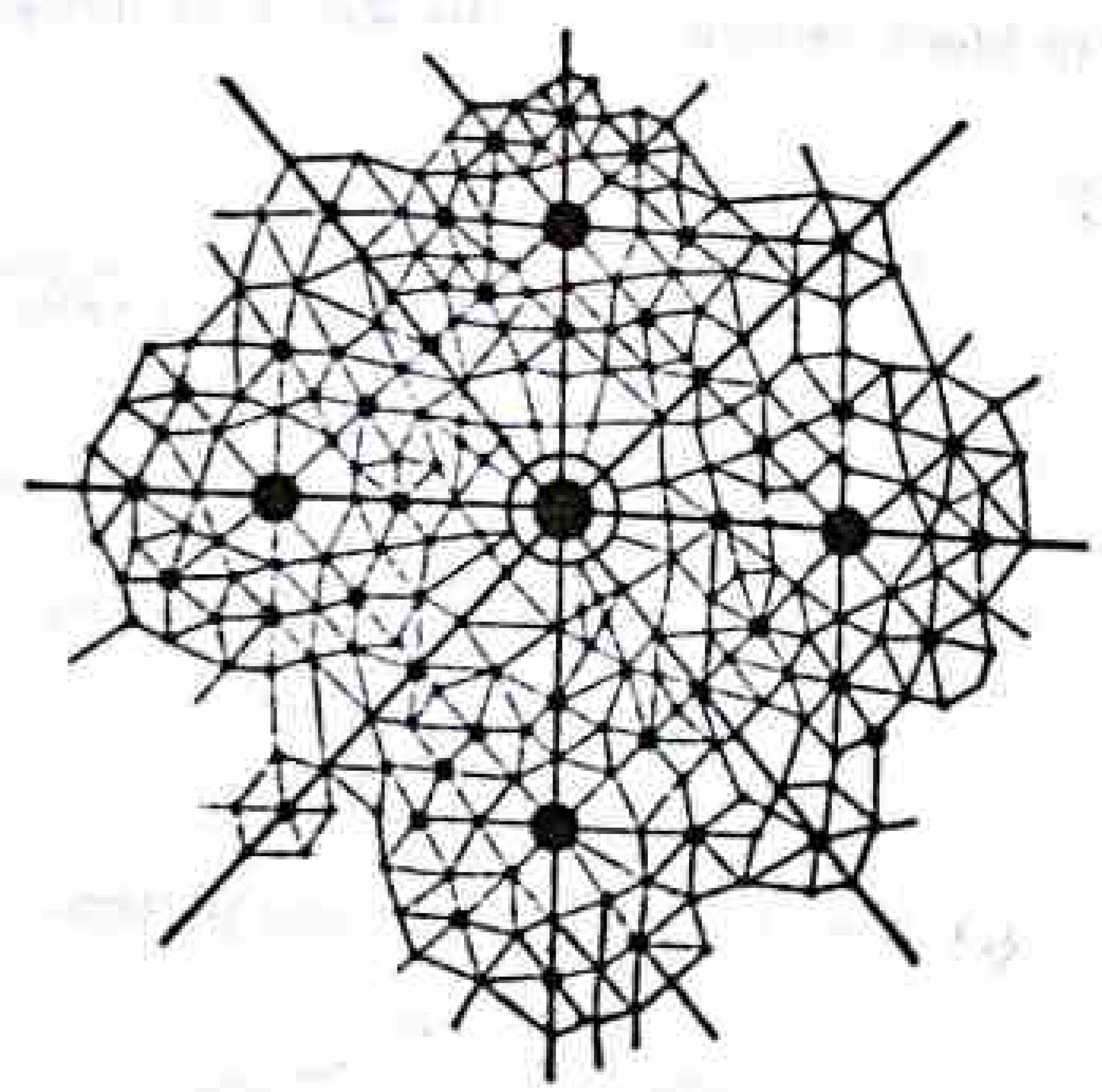


(d) radial or star and grid pattern → Nagpur

Fig. 2.5 Road Patterns *(Contd.)*



(e) Hexagonal pattern



Legend : City centre – encircled dot, sector centers - ●, Suburban centers - •, Neighbourhood centers - •, Representation of a "Minimum Travel" city (Assumed population of 2 million)

Fig. 2.5 (f) Road Patterns

Each of these patterns have their advantages and limitations. There can be a number of other geometric patterns also. The choice of the pattern very much depends on the locality, the layout of different towns, villages, industrial and production centres and on the choice of the planning engineer.

The rectangular or the block pattern has been adopted in the city roads of *Chandigarh*. But from traffic operation point this is not considered convenient. An example of radial and circular pattern is the road net work of *Connaught Place* in New Delhi. The Nagpur road plan formulae were prepared assuming *Star and Grid pattern*.

The concept of star and gird patterns has been explained below and illustrated in Fig. 2.6.

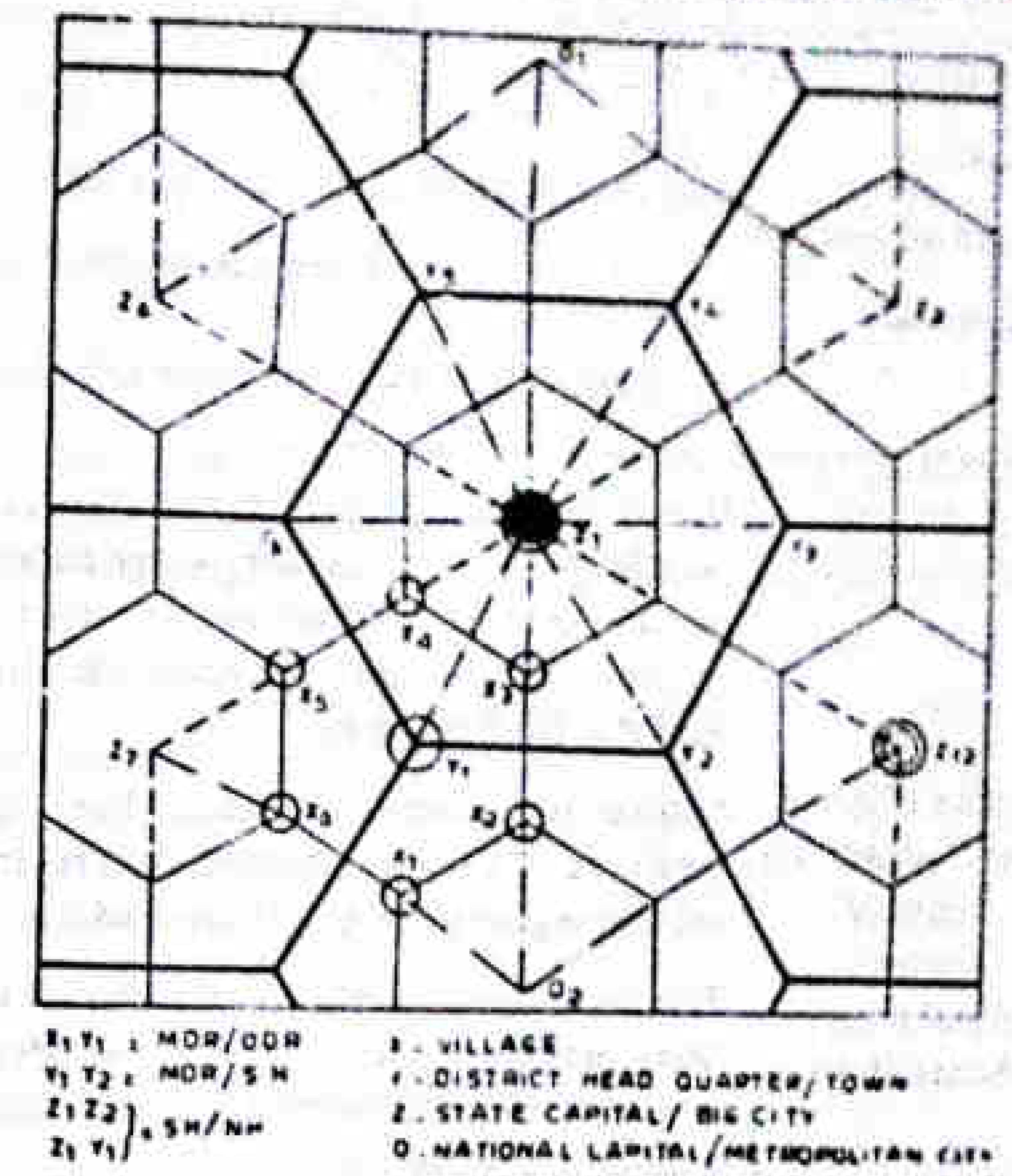


Fig. 2.6 Concept of Star and Gird Pattern

Let us assume that 'X' and 'Y' represent the villages and towns; 'Z' represent the capital towns of cities or state capitals Y_1, Y_2, Y_3 etc. are therefore acting as focal points for connecting X_1, X_2, X_3 etc., the villages. Similarly, Z_1, Z_2, Z_3 etc. are focal points for connecting Y_1, Y_2, Y_3 etc., the capital towns or cities or state capitals. Thus star and grid pattern is formed between points X_1, X_2, X_3 etc. Similarly a bigger star and grid pattern is formed with Y_1, Y_2, Y_3 etc. and Z_1, Z_2, Z_3 etc. as focal points, the whole area can thus be covered on an expanding scale. Such a network therefore, provides inter-communication facilities to each of the villages, towns, district headquarters, state capitals etc.

2.6 PLANNING SURVEYS

Highway planning phase includes

(i) Assessment of road length requirement for an area (it may be a district, state or the whole country)

(ii) Preparation of master plan showing the phasing of plan in annual and or five year plans.

Thus for assessing the road length requirement, field surveys are to be carried out to collect the data required for determining the length of the road system. The field surveys thus required for collecting the factual data may be called as planning surveys or *fact finding surveys*. The planning based on the factual data may be considered scientific and sound.

The factual studies point to an intelligent approach for planning and these studies should be carried out if the highway programme is to be protected from inconsistent and short sighted policies.

The planning surveys consist of the following studies; the details shown in Fig. 2.7.

(a) Economic studies

(b) Financial studies

(c) Traffic or road use studies

(d) Engineering studies

PLANNING SURVEYS

Economic studies	Data Collected Must Confirm To : Adequacy, Accuracy Availability, Accessibility	Population, its distribution and classification. Trend of population growth; Listing of agricultural and industrial development, their future trends-classification and listing of these details; other activities viz. Income per capita, banking, post office etc.
Financial Studies		Source of income, Revenue from taxation on road transport. Living standard, future trends-details like vehicle registration, Court fees and Local taxes etc.
Traffic or road use studies		Traffic volume, traffic flow patterns- O and D studies, Mass transportation facilities, Accidents - their costs, trends; Growth of vehicular traffic, passenger trips and good movements.
Engineering studies		Road location and alignment studies, Classification; Types of roads in use; Maintenance problems - Soil and topography studies; Road life studies.

Fig. 2.7 Details of Planning Surveys

(a) Economic Studies

The various details to be collected are useful in estimating the economics involved in the highway development programme. Hence it is desirable to find the service given by each road system to the population and products of the area. All details of the existing facilities should be available before estimating the requirement such that economic justification can be made for each plan. The details to be collected include the following :

(i) Population and its distribution in each village, town or other locality with the area classified in groups.

(ii) Trend of population growth.

(iii) Agricultural and industrial products and their listing in classified groups, area wise.

(iv) Industrial and agricultural development and future trends.

(v) Existing facilities with regard to communication, recreation and education etc.

(vi) Per capita income.

(b) Financial Studies

The financial studies are essential to study the various financial aspects like sources of income and the manner in which funds for the project may be mobilized. The details to be collected include :

(i) Sources of income and estimated revenue from taxation on road transport

(ii) Living standards

(iii) Resources at local level, toll taxes, vehicle registration and fines.

(iv) Future trends in financial aspects.

(c) Traffic or Road Use Studies

All the details of the existing traffic, their volume and pattern of flow should be known before any improvement could be planned. Traffic surveys should be carried out in the whole area and on selected routes and locations in order to collect the following particulars :

(i) Traffic volume in vehicles per day, annual average daily traffic, peak and design hourly traffic volume.

(ii) Origin and destination studies

(iii) Traffic flow patterns

(iv) Mass transportation facilities

(v) Accidents, their cost analysis and causes

(vi) Future trend and growth in traffic volume and goods traffic; trend in traffic pattern

(vii) Growth of passenger trips and the trend in the choice of modes.

(d) Engineering Studies

All details of the topography, soil and other problems such as drainage, construction and maintenance problems should be investigated before a scientific plan or programme is suggested. The studies include :

(i) Topographic surveys

(ii) Soil surveys

(iii) Location and classification of existing roads

(iv) Estimation of possible developments in all aspects due to the proposed highway development.

(v) Road life studies

(vi) Traffic-studies-Origin and Destination studies

(vii) Special problems in drainage, construction and maintenance of roads.

Thus all the above studies for collecting the factual data for highway planning are known as *fact finding surveys*. The details collected are tabulated and plotted on the maps of the area under planning.

2.7 PREPARATION OF PLANS

Before finalising the alignment and other details of the road development programme, the information collected during the fact finding surveys should be presented in the form of plans. Usually four drawings are prepared showing the various details of the area as listed below.

PLAN I General area plan showing almost all existing details viz. topography, existing road network and drainage structure, rivers, canals, nallahs etc., towns and villages with the population; commercial industrial or agricultural activities are also shown in this map.

PLAN II This plan includes the distribution of population groups in accordance with the categories made in the appropriate plan.

PLAN III This plan shows the locations of places with their respective quantities of productivity.

PLAN IV This plan shows the existing road network with traffic flows and desire lines obtained from Origin and Destination studies of traffic. Proposals received from different sources may also be shown in this plan.

2.8 INTERPRETATION OF PLANNING SURVEYS

The various details collected from the planning surveys and presented in the form of plans should be interpreted in a scientific way before arriving at the final road development programme.

The data collected could be interpreted and used for the following important purposes :

(i) To arrive at the road net-work, out of the several alternate possible systems, which has the maximum utility.

(ii) To fix up priority of the construction projects, so as to phase the road development plan of an area in different periods of time such as five year plans and annual plans.

(iii) To assess the actual road use by studying the traffic flow patterns. This data may therefore show areas of congestion which need immediate relief.

(iv) Based on the traffic type and intensity and the performance of existing types of pavement and cross drainage structures, a new structure may be designed using the data and the past experience.

(v) Comparison of the areas may be obtained on the basis of their economic activities. This information may therefore suggest the areas of immediate need for road network.

(vi) On statistical basis, the data obtained in fact finding surveys may be analysed for the future trends in development of an area i.e., growth in productivity and population which in turn generate higher traffic volume. This information may be useful in the future planning.

2.9 PREPARATION OF MASTER PLAN AND ITS PHASING

Master plan is the final road development plan for the area under study which may be a block, taluk, district, state or the whole country. Based on the above plans, different possible net work of new roads and improvement of some of the existing roads are proposed. In each proposal the population and productivity (industrial and agricultural) of each locality, the traffic flow, topography and all other details, both existing and possible changes in future are kept in view.

If some target of road length has been fixed for the country on the basis of area or population and production or both, the same may be taken as a guide for deciding the total length of the road system in each alternative proposal. In India, the target road lengths were decided by the Nagpur road plan formulae for the period 1943-63 and by the second 20-year road development plan for the period 1961-81, as mentioned in Art. 2.2.5 and 2.2.6 and also as described in Art. 2.10. These plan formulae for finding the road length are based on population and areas divide into different categories, depending on development achieved.

The next step is to compare the various alternate proposals of road systems in hand and to select the one which may be considered as best under the plan period. This is a quite difficult problem as the decision has to be a balanced one. In arriving at the best road system out of the alternate proposals, it is desirable to make use of the concept of *saturation system* based on U. S. system of highway planning.

After deciding the optimum road length for a plan period the final step is the phasing of the road development plan by fixing up the priorities for the construction of different road links.

Saturation system

In this system the optimum road length is calculated for area, based on the concept of obtaining maximum utility per unit length of road. Hence this system is called *saturation system or maximum utility system*. The factors which are taken for obtaining the utility per unit length of road are :

(a) Population served by the road network

(b) Productivity served by the net work

(i) Agricultural products

(ii) Industrial products

The following steps may be followed to find the road net work having maximum utility per unit length by the saturation system.

Step (i) Population units. Since the area under consideration may consist of villages and towns with different populations, it is required to group these into some convenient population ranges and to assign some reasonable values of utility units to each range of populations served. For example, villages having population range between 1001 and 2000 may be grouped together and be assigned one utility unit per village. Similarly the various villages and towns may be grouped in different population ranges and be assigned suitable utility units as given below :

Population less than 500, utility unit = 0.25

501 to 1000, utility unit = 0.50

1001 to 2000, utility unit = 1.00

2001 to 5000, utility unit = 2.00 etc.

From plan II of population prepared earlier, the number of towns and villages with population ranges served by each road system is found and then converted into the utility units served by each road. Thus the total number of units based on population can be obtained for each road system proposed.

Step (ii) Productivity units. The total agricultural and industrial products served by each road system should be worked out. The productivity served may be assigned appropriate values of utility units per unit weight. For example one thousand tonnes of agricultural products may be considered equivalent to one unit. Similarly the industrial products may also be assigned some suitable utility units per unit weight. However, coal, raw materials like ores etc. may be assigned lower utility values than the industrial products. From plan III showing the products in the area, the total productivity units served by each road system may be estimated.

Step (iii) Utility. The total utility units of each road system is found by adding the population units and productivity units. The total units are divided by the total road length of each system to obtain the utility rate per unit length.

Each road system having different layout and length would show different values of utility per unit length. The proposal which gives maximum utility per unit length may be chosen as the final layout with optimum road length, based on maximum utility on the saturation system.

This method is useful not only to choose the best layout from the alternate proposals, but also to phase the road development plan. The only limitation of the system is the possible variation in the relative weightages assigned to population and productivity. It is possible to give a relatively higher weightage either to the population or to a certain type of products. A sound judgment based on professional skill and experience should be helpful in providing balanced weightages for arriving at the optimum road length or the best road system with maximum utility per unit length of road.

Phasing of Road Programme

The road net-work to be constructed and improved in the plan period is decided while finalising the master plan of the road development project. The plan period may be of a long term, like that of the 20-year road plan or of shorter period like five year plans. But whatever be the plan period, it is necessary to phase the road development programme from financial considerations. In other words, it is necessary to fix up the priorities for the construction of each link of the road net work development programme to decide which link should be taken up first and which one the next and so on. The phasing may also be done for each annual budget year by fixing up the priorities.

Here again the priority for each road link may be fixed scientifically based on maximum utility. The utility per unit length of road based on population and productivity for each road is worked out. Then each link of the net work is listed in the order of priority based on utility per unit length of the road.

Example 2.1

An imaginary area with existing roads is shown in Fig. 2.8. There are four alternate plan proposals P, Q, R and S with different road length by adding extra road links to the existing roads in the area and the details of the population and products served are given below :

Prop-osal	Total road length km.	Number of towns and villages served with population range				Total agricultural & industrial products thousand tones
		1001-2000	2001-5000	5001-10000	> 10000	
P	300	160	80	30	6	200
Q	400	200	90	60	8	270
R	500	240	110	70	10	315
S	550	248	112	73	12	335

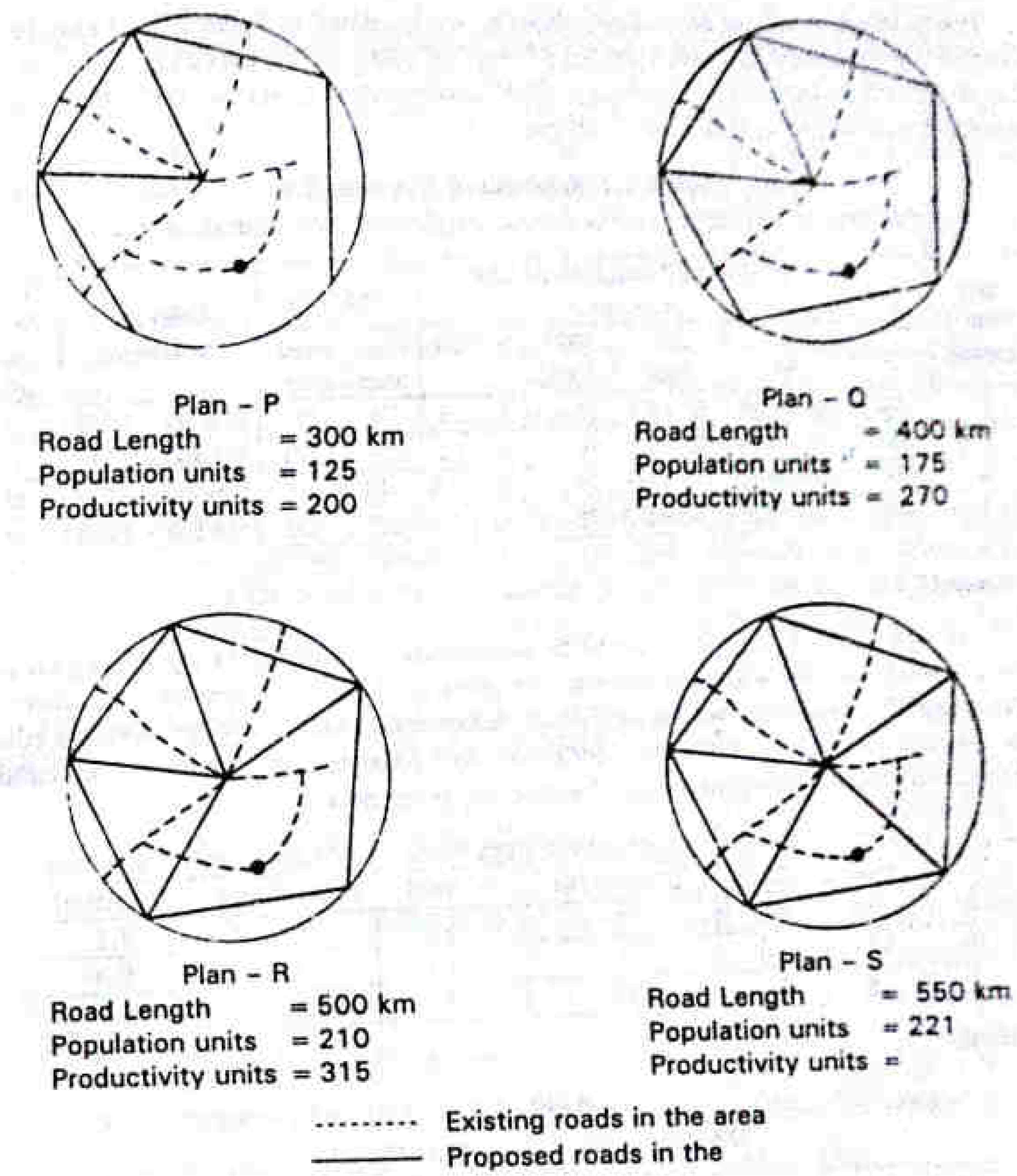


Fig. 2.8 Example 2.1

Work out the utility per unit length for each of the systems and indicate which of the plans yield the maximum utility based on saturation system.

Assume utility units as given below :

(a) Population	unit
1001-2000	0.25
2001-5000	0.50
5001-10000	1.00
> 10000	2.50
(b) Productivity	unit
1000 tonnes	1.0

Solution

The problem is solved and values obtained are tabulated in Table 2.2. It may be seen that the plan proposal Q (with total road length of 400 km) has maximum utility per unit length of road (based on population and productivity), equal to 1.12; therefore the optimum road length in this area is 400 km.

Table 2.2 Solution of Example 2.1

(Method of arriving at optimum road length based on saturation system)

Road plan proposal	Road length Km	No. of towns & villages served with population				Total units		Utility per unit length	Priority based on utility
		1001-2000	2001-5000	5001-10000	> 10000	popu-lation	produ-ctivity		
P	300	160 × 0.25	80 × 0.5	30 × 1	6 × 2.5	125	200	325/300 - 1.083	II
Q	400	200 × 0.25	90 × 0.5	60 × 1	8 × 2.5	175	270	445/400 - 1.112	I
R	500	240 × 0.25	110 × 0.5	70 × 1	10 × 2.5	210	315	525/500 - 1.050	III
S	550	248 × 0.25	112 × 0.5	73 × 1	12 × 2.5	221	335	556/550 - 1.010	IV

Example 2.2

Three new roads A, B and C are to be completed in a district during a five year plan period. Work out the order of priority for phasing the plan programme by maximum utility principle, from the data given below. Adopt utility unit of 1.0 for serving a village with population range 2000 to 5000, or for catering for 1000t of agricultural products/100t of industrial products. Assume any other data.

Road	Length km	No. of village served population			Productivity 1000 tonnes	
		< 2000	2000-5000	> 5000	Agricultural	Industrial
A	15	10	8	3	15	1.2
B	12	16	3	1	11	0.0
C	18	20	10	2	20	0.8

Solution

Table 2.3

(Phasing of road development plan)

Road	Length, km	Total utility units served by the road	Utility per unit length	Priority
A	15	10 × 0.5 + 8 ×1 + 3 × 2 + 15 × 1 + 1.2 ×10 = 46	46/15 = 3.07	I
B	12	16 × 0.5 + 3 × 1 + 1 × 2 + 11 × 1 + 0 = 24	24/12 = 2.0	III
C	18	20 × 0.5 + 10 × 1 + 2 × 2 + 20 × 1 + 0.8 × 10 = 52	52/18 = 2.89	II

Therefore order of priority is A, C and B

Assume the following utility units as per the given guide lines :

Per village served with population < 2000 = 0.5

Per village served with population 2000-5000 = 1.0

Per village served with population > 5000 = 2.0

Agricultural products served per 1000 t = 1.0

Industrial products served per 1000 t = 10.0

2.10 HIGHWAY PLANNING IN INDIA

The first attempt for proper planning of the highway development programme in India on a long term basis was made at the Nagpur Conference in 1943, as indicated in Art. 2.2.6. After the completion of the Nagpur Road Plan targets, the Second Twenty Year Plan was drawn for the period 1961-1981. The Third Twenty Year Road Development Plan for the period 1981-2001 was approved only by the year 1984.

2.10.1 Nagpur Road Plan or First 20-Year Road Plan

The Conference of Chief Engineer held at Nagpur in 1943 finalized the first twenty year road development plan for India called Nagpur Plan for the period 1943-63. The road net-work in the country was classified into five categories viz. (i) National Highways (ii) State Highways (iii) Major District Roads (iv) Other District Roads and (v) Village Roads as, explained in Art. 2.4.3.

Recommendations were made for the geometric standards of roads, bridge specifications and highway organizations. Two plan formulae were finalised at the Nagpur Conference for deciding two categories of road length for the country as a whole as well as for individual areas (like district). This was the first attempt for highway planning in India.

The two plan formulae assumed the *Star and Grid pattern* of road net work. Hence the two formulae are also called *Star and Grid Formulae.*

The total length of the first category or metalled roads for National and State Highways and Major District Roads in km is given by the formula :

$$NH + SH + MDR\ (km) = \left[\frac{A}{8} + \frac{B}{32} + 1.6N + 8T\right] + D - R \qquad (2.1)$$

where A = Agricultural area, km^2

B = Non-agricultural area, km^2

N = Number of towns and villages with population range 2001-5000.

T = Number of towns and villages with population over 5000.

D = Development allowance of 15 percent of road length calculated to be provided for agricultural and Industrial development during the next 20 years.

R = Existing length of railway track, km.

The total length of second category roads for Other District Road and Village Roads in km is given by the formula :

$$ODR + VR\ (km) = [0.32\ V + 0.8Q + 1.6P + 3.2S] + D \qquad (2.2)$$

where V = Number of villages with population 500 or less

Q = Number of villages with population range 501-1000

P = Number of villages with population range 1001-2000

S = Number of villages with population rang 2001-5000

D = Development allowance of 15 % for next 20 years.

From the above two formulae, it may be seen that in addition to the road length based on agricultural and non agricultural areas, specific road length were allocated for towns and villages of different population ranges. For example a town or village with population between 2001 and 5000 is allocated a road length of 1.6 km of first category road and 3.2 km of second category road whereas a village with population less than 500 is allotted only 0.32 km of second category road. A length of 1/8 km of first category road is provided per km^2 of agricultural area. This means that grids of first category roads are spaced at 16 kms, such that an area of 16×16 km^2 is provided with (16 + 16) or 32 km of road length, i.e., 32 km road length is available in an agricultural area of 256 km^2 or 1.0 km per 8 sq. km area and therefore the term A/8 is used in the Eq. 2.2.

Salient Features of Nagpur Road Plan

(i) The responsibility of construction and maintenance of national highways was assigned to the central government.

(ii) It was a 20-year plan intended for the period 1943-63 aiming to provide for about two lakh km of surfaced roads and remaining unsurfaced roads, so that when this target is reached, the total road length of 5, 32, 700 km with a density of about 16 km of road length per 100 sq. km area would be available in the country by the year 1963.

(iii) The formulae were based on star and gird pattern of road network. But the existing irregular pattern of roads and obligatory points not fitting in the geometric pattern were to be given due consideration.

(iv) The first category roads are meant to provide main grids bringing the farthest points in developed and agricultural area within 8 km of metalled road. The size of the gird of this category of road in agricultural area would be 16 km so that the maximum distance from the centre is 8 km and the average distance of the villages from metalled road would be less than 3.2 km. In non-agricultural area the size of the gird is of 64 km sides, the farthest distance from the centre to the metalled roads being 32 km. The length of road of this category is governed by the area, particularly the agricuitural area and towns or villages with population greater than 2001.

(v) The second category roads are meant to provide internal road system linking small villages with first category roads. The road length of second category is worked out on the basis of villages of different population ranges, of population less than 5000.

(vi) An allowance for agricultural and industrial development during the next 20 years was estimated as 15 percent and this allowance was to be provided while calculating the road length for both the categories of roads.

(vii) The length of railway tracks in the area was also considered in deciding the length of first category road. The length of railway track is directly subtracted from the estimated road length of metalled roads.

The length of various categories of roads as per the target of Nagpur Road Plan of 1943-63 and the road lengths achieved by year 1961 are given in Table 2.4. Though the achievement of total road length was higher than the plan targets, the lengths of NH and SH achieved were lesser than the plan targets.

Table 2.4 Targets of Nagpur Road Plan and Achievement by the year 1961

Sl. No.	Category of road	Nagpur Plan targets, km	Achievement by 1961, km
1.	National Highway		
	(a) NH	26,715	
	(b) National Trails	6,680	
	Total NH	33,395	22,636
2.	State Highways	86,825	62,052
3.	Major District Roads	80,145	1,13,483
	Total main roads (metalled roads)	2,00,365	1,98,171
4.	Other District Roads	1,33 ,580	1,11,961
5.	Village Roads	1,98,755	3,88,841
6.	Unclassified Roads	-	10,149
	Total	5,32,700	7,09,122

Example 2.3

The following data were collected for planning the road development programme of a backward district.

(i) Total area = 9600 km^2

(ii) Agricultural and developed area = 3200 km^2

(iii) Existing railway track length = 105 km.

(iv) Existing length of metalled road = 322 km.

(v) Existing length of unmetalled road = 450 km.

(vi) Number of towns or villages in different population ranges are as below

Population	> 5000	2001-5000	1001-2000	501-1000	< 500
Number of villages and towns	8	40	130	280	590

Calculate the additional lengths of metalled and unmetalled roads for the road system based on Nagpur Road Plan formulae for this district.

Solution

(i) The total length of metalled roads by Nagpur Plan formula is obtained from equation 2.1 and is equal to :

$$\frac{A}{8} + \frac{B}{32} + 1.6N + 8T + D - R$$

Here,

$$A = 3200 \text{ km}^2;\ B = 9600 - 3200 = 6400 \text{ km}^2$$

$$N = 40;\ T = 8;\ D = 15 \text{ percent};\ R = 105 \text{ km}$$

$$\text{Metalled road length} = \left[\frac{3200}{8} + \frac{6400}{32} + 1.6 \times 40 + 8 \times 8\right] + 15\% \text{ of total road length} - 105$$

$= [400 + 200 + 64 + 64] + 15\% \text{ of RL} - 105$

$= 728 + \frac{15 \times 728}{100} - 105 = 732.2$ km

Additional metalled road needed

$= 732.2 - 322 = 410.2$ km

(ii) Total length of unmetalled roads by Nagpur plan formula may be obtained from equation 2.2 and is equal to :

$$[0.32\ V + 0.8\ Q + 1.6\ P + 3.2\ S] + D$$

Here,

$V = 590, Q = 280, P = 130, S = 40, D = 15\%$

Unmetalled road length

$= [0.32 \times 590 + 0.8 \times 280 + 1.6 \times 130 + 3.2 \times 40] + 15\%$ road length

$= [188.8 + 224 + 208 + 128] + 15\%$ of RL

$= 748.8 + \frac{748.8 \times 15}{100} = 861$ km

Additional unmetalled roads required

$= 861 - 450 = 411$ km

2.10.2 Second Twenty Year Road Plan (1961-81)

The Nagpur road plan was intended for the period 1943-63, but the target road length was nearly completed earlier in 1961 (as shown in Table 2.4), mainly because of the phased development that took place in the country during the first two 5-year plans. Hence the next long term plan for the twenty year period commencing from 1961 was initiated by the IRC and was finalised by the sub committee and this was approved by the Chief Engineers. *The Second Twenty Year Road Development Plan 1961-81* is also called *Bombay Road Plan.*

The second road plan envisaged overall road length of 10,57,330 km by the year 1981. The cost of the plan has been worked out to Rs. 5,200 crores based on 1958 price level for a period of 20 years from 1961.

Five different formulae were framed to calculate the lengths of National Highways, State Highways, Major District Roads, Other District Roads and Village Roads.

These five formulae are given below :

(i) National Highways (km)

$$= \left[\frac{A}{64} + \frac{B}{80} + \frac{C}{96}\right] + [32K + 8M] + D \qquad (2.3)$$

(ii) National Highways + State Highways (km)

$$= \left[\frac{A}{20} + \frac{B}{24} + \frac{C}{32}\right] + [48K + 24M + 11.2N + 1.6P] + D \qquad (2.4)$$

(iii) National Highways + State Highways + Major District Roads (km)

$$= \left[\frac{A}{8} + \frac{B}{16} + \frac{C}{24}\right] + [48K + 24M + 11.2N + 9.6P + 6.4Q + 2.4R] + D \qquad (2.5)$$

(iv) National Highways + State Highways + Major District Roads + Other District Roads (km)

$$= \left[\frac{3A}{16} + \frac{3B}{32} + \frac{C}{16}\right] + [48K + 24M + 11.2N + 9.6P + 12.8Q + 4R + 0.8S + 0.32T] + D \qquad (2.6)$$

(v) National Highways + State Highways + Major District Roads + Other District Roads + Village Roads i.e., all roads (km)

$$= \left[\frac{A}{4} + \frac{B}{8} + \frac{C}{12}\right] + [48K + 24M + 11.2N + 9.6P + 12.8Q + 5.9R + 1.6S + 0.64T + 0.2V] + D \qquad (2.7)$$

where A = Developed and agricultural areas; km^2

B = Semi-developed area, km^2

C = Undeveloped area, km^2

K = Number of towns with population over 1,00,000

M = Number of towns with population range 1,00,000-50,000

N = Number of towns with population range 50,000-20,000

P = Number of towns with population range 20,000-10,000

Q = Number of towns with population range 10,000-5,000

R = Number of towns with population range 5,000-2,000

S = Number of towns with population range 2,000-1,000

T = Number of towns with population range 1,000-500

V = Number of towns with range below 500

D = Development allowance of 5 percent of road length calculated for further development and other unforeseen factors.

Salient features of the Second 20-year Plan (1961-81)

(i) This plan is considered to be drawn more scientifically in view of development needed in under-developed areas. The target of this plan is to provide a total road length of 32 km per 100 sq. km area which is almost double of that achieved up to the year 1961.

(ii) Maximum distance of any place in a developed or agricultural area would be 6.4 km from a metalled road and 2.4 km from any category of roads.

(iii) The maximum distance from any place in a semi-developed area would be 12.8 km from a metalled road and 4.8 km from any road; similarly the maximum distance in an undeveloped area would be 19.2 km from a metalled road and 8.0 km from any road.

(iv) Every town with population above 2000 in plains and above 1000 in semi-hilly areas and above 500 in hilly areas should be connected by a metalled road.

(v) While calculating road length in hilly regions, an allowance up to 100 percent may be made in arriving at the road length. Hills with altitude above 2300 metres may be ignored in calculating road length in view of thin population.

(vi) Expressways have also been considered in this plan and 1600 km of length has been included in the proposed target of National Highways.

(vii) Length of railway track is considered independent of the road system and hence it is not subtracted to get the road length.

(viii) The development factor of only 5 percent is provided for future development and unforeseen factors.

Comparison of Nagpur Plan & Second 20-year Road Plan or Bombay Road Plan

(i) Nagpur road plan gives two formulae, one is to find the length of first category roads or metalled roads consisting of National Highways, State Highways and major District Roads; the second formula is to find the length of second category roads or unmetalled roads consisting of other District Roads and Village Roads. Hence it is not possible to get the road length for each class of the road separately. In the second 20-year plan (1961-81), five different formulae have been given from which the length of each class of road i.e., NH, SH, MDR, ODR and VR, could be obtained individually.

(ii) The Nagpur road plan divides the area into two parts, viz. agricultural and non-agricultural area. In Bombay road plan, the area is divided into three parts.

(1) developed and agricultural area

(2) semi-developed area and

(3) undeveloped and uncultivated area

(iii) The second 20-year plan has a target road length of 32 km per 100 sq. km area which is double the Nagpur plan target.

(iv) Nagpur road plan formula does not take into account the towns with very large population. First category road length is decided based on towns with population range 2001-5000 and those with population above 5000, thus grouping all larger towns with population higher than 5000 together. But at present there are large number of towns having population varying from few thousands to several lakhs. It appears that such high growth of population in towns was not anticipated while the Nagpur road plan was drawn in 1943. In second 20-year plan, towns have been divided into nine different population ranges from less than 500 for the smallest town or villages to above 1,00,000 for larger towns.

(v) Nagpur road plan allowed deduction of the length of railways track in the area while calculating the length of first category roads. But it has been realised later that the highway system should be able to develop independently and so in the Bombay Road Plan, the length of railway track is not deducted.

(vi) Allowance for development of agriculture and industry during the next 20 years was made in Nagpur plan by allowing 15 percent increase in the calculated road length of both categories. The allowance for development due to unforeseen factors according to the second plan is only 5 percent.

(vii) The 1961-81 plan has provided 1600 km of expressways out of the proposed National Highways.

The lengths of various categories of roads as per the targets of Bombay Road Plan of 1961-81 and the road lengths achieved by the year 1981 are given in Table 2.5. The lengths of NH, SH and MDR achieved by the year 1981 fell short of the Plan targets. Further there were considerable deficiencies in geometric design standards, pavement surfacing and bridging requirements specified in the Plan. The lengths of ODR and VR achieved were however much higher. Therefore the total road length achieved by the year 1981 was higher than the targeted total length, resulting in a overall road density of 45.7 km of road length per 100 sq. km area in the country (including unclassified roads) as against the targeted density of about 32 km per 100 sq. km area in the Plan.

Table 2.5 Targets of Bombay Road Plan and Achievements by the Year 1981

Category of Road	Bombay Plan targets, km	Road length by 1981, km
National Highways	51,500	31,737
State Highways	1,12,650	95,491
Major District Roads	2,41,400	1,53,000
Other District Roads	2,89,680	-
Village Roads	3,62,100	-
Total ODR and VR	6,51,780	9,12,684
Total or NH, SH, MDR, ODR and VR	10,57,330	11,92,912
Unclassified roads such as urban roads and project roads	-	3,09,785
Grand total of all categories	10,57,330	15,02,697

Example 2.4

Calculate the lengths of National and State highways required in a district with a total area of 7200 km^2, developed, semi-developed and undeveloped areas being 30, 45 and 25% of the district. The number of towns with population over 1.0, 0.5-1.0, 0.2-0.5 and 0.1 - 0.2 lakhs are 3, 7, 12 and 20 respectively in the district. Use the formulas :

$$NH = [A/64 + B/80 + C/96 + 32K + 8M] + D$$

$$NH + SH = [A/20 + B/24 + C/32 + 48K + 24M + 11.2N + 1.6P] + D$$

Solution

$A = 7200 \times 0.30 = 2160$ km^2;

$B = 7200 \times 0.45 = 3240$ km^2;

$C = 7200 \times 0.25 = 1800$ km^2;

$K = 3; M = 7; N = 12;$

$P = 20; D = 5\%$ of road length

$$NH = \left[\frac{2160}{64} + \frac{3240}{80} + \frac{1800}{96} + 32 \times 3 + 8 \times 7\right] + 5\% \text{ of RL}$$

$$= 245 \times 10.5 = 257.25 \text{ km}$$

$$NH + SH = \left[\frac{2160}{20} + \frac{3240}{24} + \frac{1800}{32} + 48 \times 3 + 24 \times 7 + 11.2 \times 12 + 1.6 \times 20\right] + 5\% \text{ of RL}$$

$$= 777.65 \times 1.05 = 816.53 \text{ km}$$

$$SH = 816.53 - 257.25 = 559.28 \text{ km}$$

Example 2.5

Calculate the total lengths of NH, SH, MDR, ODR and VR needed in a district as per second 20-year road development plan or Bombay Road Plan. The data collected from the district are given below :

(i) Total area = 18,400 km^2

(ii) Developed and agricultural area = 8000 km^2

(iii) Undeveloped area = 4800 km^2

(iv) Population centres are as given below :

Population range	Number of towns
< 500	200
500 - 1000	350
1000 - 2000	750
2000 - 5000	360
5000 - 10,000	150
10,000 - 20,000	80
20,000 - 50,000	25
50,000 - 1,00,000	10
> 1,00,000	5

Solution

(i) $$NH (km) = \frac{A}{64} + \frac{B}{80} + \frac{C}{96} + [32K + 8M] + D$$

Here, A = 8,000 km^2; C = 4,800 km^2

B = 18,400 – [8,000 + 4,800] = 5,600 km^2

K = 4; M = 10; D = 5%

$$NH (km) = \left[\frac{8000}{64} + \frac{5600}{80} + \frac{4800}{96} + 32 \times 5 + 8 \times 10\right] + 5\% \text{ of RL}$$

$$= [125 + 70 + 50 + 160 + 80] \frac{105}{100} = 485 \times \frac{105}{100} = 509.25$$

(ii) $$NH + SH (km) = \left[\frac{A}{20} + \frac{B}{24} + \frac{C}{32} + 48K + 24M + 11.2N + 1.6P\right] + D$$

Here, N = 25; P = 80

$$NH + SH = \left[\frac{8000}{20} + \frac{5600}{24} + \frac{4800}{32} + 48 \times 5 + 24 \times 10 + 11.2 \times 25 + 1.6 \times 80\right] + 5\% \text{ of RL}$$

$$= [400 + 233 + 150 + 240 + 240 + 280 + 128] \frac{105}{100}$$

$$= 1671 \times \frac{105}{100} = 1754.5 \text{ km}$$

$$SH = 1754.5 - 509.25 = 1245.25 \text{ km}$$

(iii) NH + SH + MDR (km) =

$$\left[\frac{A}{8} + \frac{B}{16} + \frac{C}{24} + 48K + 24M + 11.2N + 9.6P + 6.4Q + 2.4R\right] + D$$

Here, Q = 150; R = 360

$$NH + SH + MDR = \left[\frac{8000}{8} + \frac{5600}{16} + \frac{4800}{24} + 48 \times 5 + 24 \times 10 + 11.2 \times 25 + 9.6 \times 80 + 6.4 \times 150 + 2.4 \times 360\right] + 5\% \text{ of RL}$$

$$= [1000 + 350 + 200 + 240 + 240 + 280 + 768 + 960 + 864] \frac{105}{100}$$

$$= 4902 \times \frac{105}{100} = 5147 \text{ km}$$

$$MDR = 5147 - 1754.5 = 3392.5 \text{ km}$$

(iv) $$NH + SH + MDR + ODR (km) = \frac{3A}{16} + \frac{3B}{32} + \frac{C}{16} + 48K + 24M + 11.2N + 9.6P + 12.8Q + 4R + 0.8S + 0.32T + D$$

Here S = 750; T = 350

$$NH + SH + MDR + ODR = \left[\frac{3 \times 8000}{16} + \frac{3 \times 5600}{32} + \frac{4800}{16} + 48 \times 5 + 24 \times 10 + 11.2 \times 25 + 9.6 \times 80 + 12.8 \times 150 + 4 \times 360 + 0.8 \times 750 + 0.32 \times 350\right] + 5\% \text{ of RL}$$

$$= [1500 + 525 + 300 + 240 + 240 + 280 + 768 + 1920 + 1440 + 600 + 112] \frac{105}{100}$$

$$= 7925 \times \frac{105}{100} = 8321.2 \text{ km}$$

$$ODR = 8321.2 - 5147.0 = 3174.2 \text{ km}$$

(v) NH + SH + MDR + ODR + VR (km) =

$$\left[\frac{A}{4} + \frac{B}{8} + \frac{C}{12} + 48K + 24M + 11.2N + 9.6P + 12.8Q + 5.9R + 1.6S + 0.64T + 0.2V\right] + D$$

$$= \left[\frac{8000}{4} + \frac{5600}{8} + \frac{4800}{12} + 48 \times 5 + 24 \times 10 + 11.2 \times 25 + 9.6 \times 80 + 12.8 \times 150 + 5.9 \times 360 + 1.6 \times 750 + 0.64 \times 350 + 0.2 \times 200\right] + 5\% \text{ of RL}$$

$$= [2000 + 700 + 400 + 240 + 240 + 280 + 768 + 1920 + 2124 + 1200 + 224 + 40]\frac{105}{100}$$

$$= 10136 \times \frac{105}{100} = 10642.8 \text{ km}$$

Here $V = 200$

$VR = 10{,}642.8 - 8321.2 = 2321.6$ km

The various road length required are as follows :

NH = 509.25 km;

SH = 1245.25 km;

MDR = 3392.50 km;

ODR = 3174.2 km;

VR = 2321.6 km

2.10.3 Third Twenty Year Road Development Plan 1981-2001

Policies and Objectives :

As mentioned in Art. 2.2.8, the Third Twenty Year Road Development Plan, 1981-2001 (also known as *Lucknow Road Plan*) was finalised and the plan document was published by the year 1984. The major policies and objectives of this road plan are listed below :

(i) The future road development should be based on the revised classification of road system consisting of primary, secondary, and tertiary road systems as mentioned in Art. 2.4.4.

(ii) The road net work should be developed so as to preserve the rural oriented economy and to develop small towns with all the essential facilities. All the villages with population over 500 (based on 1981 census) should be connected by all weather roads by the end of this century.

(iii) The overall road density in the country should be increased to 82 km per 100 sq. km area by the year 2001. The corresponding values of planned road densities are 40 for hill areas of altitude upto 2100 metres above MSL and 15 km per sq. km area for altitude above 2100 metre.

(iv) The National Highway net work should be expanded to form square grids of 100 km sides so that no part of the country is more than 50 km away from a NH.

(v) The lengths of SH and MDR required in a state or region should be decided based on both areas and number of towns with population above 5,000 in the state or region.

(vi) Expressways should be constructed along major traffic corridors to provide fast travel.

(vii) All the towns and villages with population over 1500 should be connected by Major District Roads and the villages with population 1000 to 1500 by Other District Roads. There should be a road within a distance of 3.0 km in plains and 5.0 km in hilly terrain connecting all villages or groups of villages with population less than 500.

(viii) Roads should also be built in less industrialized areas to attract the growth of industries.

(ix) Long term master plans for road development should be prepared at various levels, i.e., taluk, district, state and national levels. The road net work should be scientifically decided to provide maximum utility.

(x) The existing roads should be improved by rectifying the defects in the road geometrics, widening of the pavements, improving the riding quality of the pavement surface and strengthening of the pavement structure to save vehicle operation cost and thus to conserve energy.

(xi) There should be improvements in environmental quality and road safety.

Determination of Road Lengths by Third Road Plan Formulae

I. Primary Road System

(i) Expressways of total length 2000 km to be developed for fast travel based on traffic requirements.

(ii) National Highways are to be based on the concept of 100 km square grids, by providing 100 + 100 = 200 km of road length per 100 × 100 = 10000 sq. km area i.e., one km per 50 sq. km area. Therefore the total length of NH in the country or in a state could be obtained by dividing the total area of the country by 50

Length of NH in the country, km = 32,87,782/50 = 65,756 km, say 66,000 km

*II. Secondary System :*Length of SH

The roads consisting of NH and SH should pass through every town or urban area; there are 3364 such towns in the country as defined by 1981 census. Therefore the area of each of the square grids would be equal to total area divided by the number of towns = 32,87,782/3364 = 977.3 sq. km, with sides $\sqrt{977.3}$ = 31.26 km. Therefore the length of (NH + SH) will be 2 × 31.26 = 62.5 km for each such square gird. Thus the total of (NH + SH) for 3364 towns in the country = 62.5 × 3364 = 2,10,250 km. The total length of NH for the country as determined earlier is 66,000 km. Thus the total length of SH in the country = 2,10,250 − 66,000 = 1,44,250 km, or say 1,45,000 km.

The total length of SH required for any State may be determined from the following two relations :

(a) By total area, SH, km = Area of the State, sq. km/25

(b) By total no. of towns and area in the State, SH, km = 62.5 × no. of towns in the State − area of the State, sq. km/50.

(iii) Length of MDR

Total length of MDR in the country has been worked out as 3,00,000 km.

The total length of MDR required in a State is determined from one of the following formulas :

(a) By total area, MDR, km = Area of the State, sq. km/12.5.

(b) By no. of towns in the State, MDR, km = 90 × no. of towns in the State.

III. Tertiary System or Rural Roads : Length of ODR and VR.

The total length of ODR and VR in the country by the year 2001 as per the third road development plan shall be 21,89,000 km so that the overall length of all categories of roads i.e., NH, SH, MDR, ODR and VR will be 27 lakh km by the end of the plan period. The total length of all the above five categories of roads (excluding urban roads) as per this plan target have been worked out for each State separately; the targeted total road lengths of some of the States in India are given in Table 2.6. The length of Rural Roads consisting of ODR and VR in each State may be determined by subtracting the length of (NH + SH + MDR) calculated for each state as mentioned in the previous paragraphs, from the total road length given in Table 2.6. The distribution of the lengths of ODR and VR is to be decide by each state depending on the requirement.

Table 2.6 Estimated total road lengths in some states by the year 2001, as per third road development plan targets

State	Total road length (NH, SH, MDR, ODR & VR) by the year 2001, km
1. Andhra Pradesh	1,74,856
2. Assam	85,282
3. Bihar	2,12,032
4. Gujarat	1,14,886
5. Haryana	34,738
6. Himachal Pradesh	61,262
7. Jammu & Kashmir	53,840
8. Karnataka	1,44,654
9. Kerala	1,28,963
10. Madhya Pradesh	3,41,268
11. Maharashtra	2,20,877
12. Orissa	1,65,507
13. Punjab	50,132
14. Rajasthan	2,09,392
15. Tamil Nadu	1,38,115
16. Uttar Pradesh	3,55,160
17. West Bengal	1,15,792

Example 2.6

The area of a certain district in India is 13,400 sq. km and there are 12 towns as per 1981 census. Determine the lengths of different categories of roads to be provided in this district by the year 2001.

Solution

(i) Length of NH, km = 13400/50 = 168 km

(ii) Length of SH :

(a) By area, SH, km = 13400/25 = 536 km

(b) By area and no. of towns, SH, km = 62.5 × 12 – 13400/50 = 482 km

Adopt length of SH (higher of the two criteria) = 536 km

(iii) Length of MDR, in the District :

(a) By area, MDR, km = 13400/12.5 = 1072 km

(b) By no. of towns, MDR, km = 90 × 12 = 1080 km

Provide length of MDR (higher of the two criteria) = 1080 km

(iv) Total length of all categories of roads may be assumed to provide an overall density of road length equal to 82 km per 100 sq. km area by the year 2001.

NH + SH + MDR + ODR + VR = 13400 × 82/100 = 10988 km

Length of NH + SH + MDR = 268 + 536 + 1080 = 1884 km

Therefore length of Rural roads consisting of ODR + VR = 10988 – 1884 = 9104 km

(i) Primary system of NH = 268 km

(ii) Secondary system consisting of SH = 536 and MDR = 1080, total length = 1616 km

(iii) Tertiary system of Rural Roads consisting of ODR and VR = of length = 9104 km

(iv) Total road length = 10,988 km

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PROBLEMS

1. Briefly discuss the historical development of road construction.

2. What are the salient features of early Roman Roads ? How do these differ from the present day road construction ?

3. Compare the construction methods of Telford and Macadam; bring out the points of differences.

4. Briefly explain the Macadam's method of road construction. Why is this method considered better and more scientific compared to the previous methods ?

5. Discuss briefly with reasons how further improvements to the water bound macadam road became necessary.

6. Briefly outline the highway development in India.

7. Explain the necessity and objects of highway planning.

8. What are the various methods of classifying roads ? Briefly outline the classification based on location and function as suggested in the Nagpur Road Plan.

9. What are the significant recommendations of Jayakar Committee Report ? Mention how this helped in road development in India ?

10. Explain briefly the modified classification of road system in India as per the Third Twenty Year Road Development Plan, 1981-2001.

11. Briefly outline the main features of various road patterns commonly in use. Explain with sketches the star and grid pattern.

12. What are the various surveys to be carried out before planning a highway system for a given area ? Explain briefly.

13. What are the various plans to be prepared after the planning surveys are carried out ?

14. What are the uses of fact finding surveys ? How are these used and interpreted ?

15. Explain how the master plan is prepared and the road development programme is phased.

16. Explain why the saturation system is considered a rational method to decide the final road network and for phasing the road development programme. Illustrate the saturation system with an example.

17. There are five alternate proposals of road plans for a backward district. The details are given below. Justify with reasons which proposal is the best assuming, Utility units of 0.5, 1.0, 2, 4 and 8 for the five population ranges and 1.0 and 5 per 1000 t of agricultural and industrial products served.

Proposal	Total road length, km	Number of towns and villages served with population range					Productivity in thousand tonnes	
		<2000	2001-5000	5001-10000	10001-20000	>20000	Agricultural	Industrial
P	500	100	150	40	20	3	150	20
Q	600	200	250	68	28	3	220	25
R	700	270	350	82	36	4	300	35
S	800	280	410	91	41	4	400	42
T	900	290	430	96	44	4	430	45

[Ans. : Proposal S with 1.923 units/km is the best; R = 1.857,
T = 1.806, Q = 1.612, P = 1.268]

18. Four new road links A, B, C and D are to be constructed during a five-year plan period. Suggest the order of priority for phasing the road construction programme based on maximum utility approach. Assume utility units of 0.5, 1.0, 2 and 4 for the four population ranges and 2, 2 and 5 units per 1000 t of agricultural, raw material and industrial products from the following data :

Road link	Length km	No. of villages served with population range				Productivity served, t		
		< 500	501-1000	1001-2000	> 2000	Agricultural	Raw Material	Industrial product
A	75	30	15	10	3	8000	3000	1000
B	35	20	8	6	3	5000	1000	1600
C	40	15	6	5	5	6000	2000	3200
D	50	40	4	3	2	3000	7000	500

[Ans.: I Priority = C (1,888 units/km), II = B (1.771), III = A (1.453), IV = D (1.21)]

19. What is the importance of Nagpur road plan in highway planning of our country ? Explain the plan formulae and the salient features of the plan.

20. Discuss the second twenty year road plan of 1961-81 and its salient features.

21. Compare the Nagpur road plan and the second twenty year road plan; discuss the merits of each.

22. From the following data for a district, calculate the road length required based on Nagpur road plan.

Total area = 6300 km^2

Agricultural area = 2800 km^2

No. of villages with population ranges < 500, 501-1000, 1001-2000, 2001-5000, and above 5001 are 450, 320, 110, 50 and 10 respectively. Length of railway track = 75 km.

Population range of towns and villages	Number of towns and village
2001 - 5000	120
5001 - 10,000	35
10,001 - 20,000	20
20,001 - 50,000	10
50,001 - 100,001	6
> 100,001	2

[Ans.: I category roads = 637.3 and II category roads = 846.4 km]

23. Work out the lengths of NH, SH and MDR required in a district by second 20-year road development plan (1961-81) using the following plan formulas and data of Problem 22 :

$$NH = A/64 + B/80 + C/96 + 32K + 8M + D$$

$$NH + SH = A/20 + B/24 + C/32 + 48K + 24M + 11.2N + 1.6P + D$$

$$NH + SH + MDR = 3A/16 + 3B/32 + C/16 + 48K + 24M + 11.2N + 9.6P + 12.8Q + 4R + 0.8S + 0.32T + D$$

Area of the district = 10,800 km^2

Developed and agricultural area = 4100 km^2

Undeveloped area = 2300 km^2

[Ans. : NH = 267.8, SH = 618.6, MDR = 1149.9 km]

24. What are the policies and goals of the Third Road Development Plan for 1981-2001 ?

25. Explain how the road lengths of different categories for a state are determined for the year 2001 using the Third Road Development Plan concept.

26. Determine the length of different categories of roads in a state in India by the year 2001, using the Third Road Development formula and the following data :

Total area of the state = 80,000 sq. km

Total no. of towns as per 981 census = 86

Overall road density aimed at = 82 km per 100 sq. km area

[Ans.: Primary/NH = 1600, Secondary : SH = 3200, MDR = 7740, Tertiary : ODR & VR = 52,485, Total = 65,600 km]

27. Write short notes on :

(a) Central road fund

(b) Nagpur road plan

(c) Fact finding surveys

(d) Master plan

(e) Saturation system

(f) Star and grid pattern

(g) Indian Roads Congress

(h) Jayakar Committee

Chapter 3

Highway Alignment and Surveys

3.1 HIGHWAY ALIGNMENT

The position or the layout of the centre line of the highway on the ground is called the alignment. The horizontal alignment includes the straight path, the horizontal deviations and curves. Changes in gradient and vertical curves are covered under vertical alignment of roads.

A new road should be aligned very carefully as improper alignment would result in one or more of the following disadvantages :

(a) increase in construction cost

(b) increase in maintenance cost

(c) increase in vehicle operation cost

(d) increase in accident rate.

Once the road is aligned and constructed, it is not easy to change the alignment due to increase in cost of adjoining land and construction of costly structures by the road side. Hence the importance of careful considerations while finalizing the alignment of a new road need not be over emphasised.

3.1.1 Requirements

The basic requirements of an ideal alignment between two terminal stations are that it should be :

(a) short

(b) easy

(c) safe, and

(d) economical

Short : It is desirable to have a short (or shortest) alignment between two terminal stations. A straight alignment would be the shortest, though there may be several practical considerations which would cause deviations from the shortest path.

Easy : The alignment should be such that it is easy to construct and maintain the road with minimum problems. Also the alignment should be easy for the operation of vehicles with easy gradients and curves.

Safe : The alignment should be safe enough for construction and maintenance from the view point of stability of natural hill slopes, embankment and cut slopes and foundation of embankments. Also it should be safe for the traffic operation with safe geometric features.

Economical : The road alignment could be considered economical only if the total cost including initial cost, maintenance cost and vehicle operation cost is lowest. All these factors should be given due consideration before working out the economics of each alignment.

The alignment should be such that it would offer maximum utility by serving maximum population and products. The utility of a road should be judged from its utility value per unit length of road. (For details refer Art. 2.9).

3.1.2 Factors Controlling Alignment

For an alignment to be shortest, it should be straight between the two terminal stations. This is not always possible due to various practical difficulties such as intermediate obstructions and topography. A shortest route may have very steep gradients and hence not easy for vehicle operation. Similarly, there may be construction and maintenance problems along a route which may otherwise be short and easy. Roads are often deviated from the shortest route in order to cater for intermediate places of importance or obligatory points.

A road which is economical in the initial construction cost, need not necessarily be the most economical in maintenance or in vehicle operation cost. It may also happen that the shortest and easiest route for vehicle operation may work out to be the costliest of the different alternatives from construction view point. Thus it may be seen that an alignment can seldom fulfill all the requirements simultaneously; hence a judicial choice is made considering all the factors.

The various factors which control the highway alignment in general may be listed as :

(a) Obligatory points

(b) Traffic

(c) Geometric design

(d) Economics

(e) Other considerations

In hill roads additional care has to be given for :

Stability

Drainage

Geometric standards of hill roads, and

Resisting length

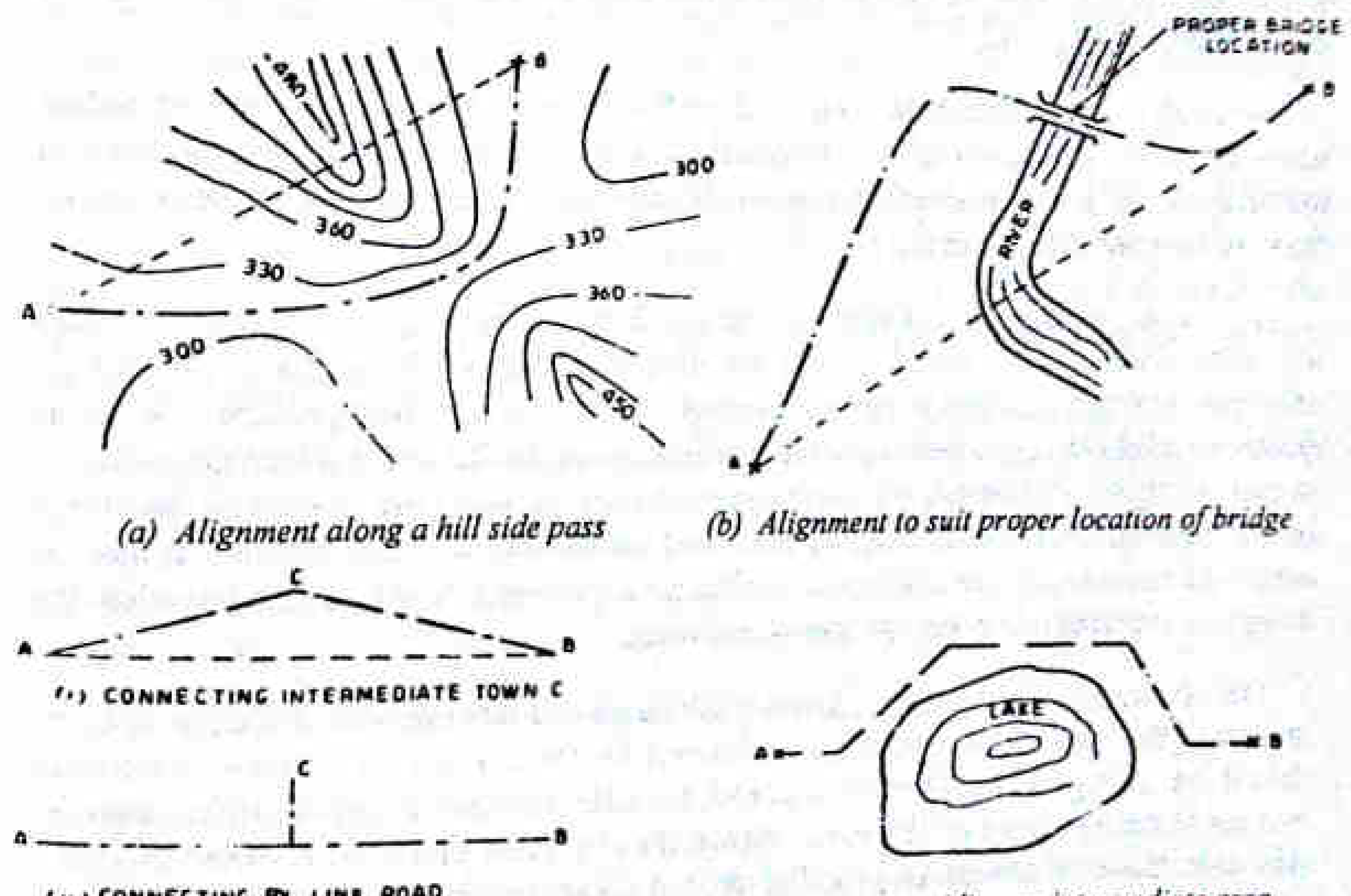
(a) *Obligatory Points* : There are control points governing the alignment of the highways. These control points may be divided broadly into two categories.

(i) Points through which the alignment is to pass.

(ii) Points through which the alignment should not pass.

(i) Obligatory points through which the road alignment has to pass may cause the alignment to often deviate from the shortest or easiest path. The various examples of this category may be bridge site, intermediate town, a mountain pass or a quarry.

When it is necessary to cross hill range, mountains or high ridges the various alternatives are to cut a tunnel across or to go round the hills or to deviate until a suitable hill pass is available. The suitability of these alternatives depend on many other factors, like the topography and site conditions and cost considerations. Figure 3.1 a shows how the straight alignment AB is deviated along the hill side pass, thus avoiding a tunnel or heavy cutting.



(a) *Alignment along a hill side pass* (b) *Alignment to suit proper location of bridge*

(c) *Alignment to connect intermediate town* (d) *Alignment avoiding an intermediate area*

Fig. 3.1 Obligatory Points Controlling Alignment of Roads

The road bridge across a river can be located only at place where the river has straight and permanent path and where the bridge abutment and pier can be properly founded. The road approaches to this bridge should not be curved near the bridge and as far as possible the skew crossing should be avoided. Thus in order to locate a bridge across a river the alignment may have to be changed. Figure 3.1b shows that the straight alignment between stations A and B which passes across the river bend, is to be deviated along the path shown in order to cross the river at a proper bridge location at the straight portion of the river on the up-stream side of the band.

While aligning a road between two stations, it may often be desirable to connect some of the important intermediate towns, villages or other places. The straight alignment AB may be shifted along line ABC, as shown in Fig. 3.1c in order to connect the intermediate station C. It is also possible to connect the station C with a link road as shown in the same figure, thus avoiding the deviation of the straight alignment.

(ii) Obligatory points through which the road should not pass also may make it necessary to deviate from the proposed shortest alignment. The obligatory points which should be avoided while aligning a road include religious places, very costly structures, unsuitable land etc. Religious places like temple, mosques, church, grave or tomb have been protected by the law from being acquired for any purpose. Acquiring costly structures would mean heavy compensation resulting in increased cost. Marshy, peaty and water logged areas are generally unsuitable for road construction and should be avoided as far as possible. However if there, is no alternative and the alignment has to be taken across such an area, the construction and maintenance costs are likely to be very high due to special construction techniques and drainage measures to be adopted.

A lake, a pond or a valley which falls on the path of a straight alignment will also necessitate the alignment to deviate from the straight path and go round along the grade line as shown in Fig. 3.1d.

(b) *Traffic* : The alignment should suit traffic requirements. Origin and Destination study should be carried out in the area and the *desire lines* be drawn showing the trend of traffic flow. The new road to be aligned should keep in view the desired lines, traffic flow patterns and future trends.

(c) *Geometric Design* : Geometric design factors such as gradient, radius of curve and sight distance also would govern the final alignment of the highway. If straight alignment is aimed at, often it may be necessary to provide very steep gradients. As far as possible while aligning a new road, the gradient should be flat and less than the *ruling* or design gradient. Thus it may be necessary to change the alignment in view of the design speed, maximum allowable superelevation and coefficient of lateral friction. It may be necessary to make adjustment in the horizontal alignment of roads keeping in view the minimum radius of curve and the transition curves.

The absolute minimum *sight distance*, which should invariably be available in every section of the road, is the safe stopping distance for the fast moving vehicles. Also there should be enough distance visible ahead for safe overtaking operations of vehicles moving at design speed on the road. Hence the alignment should be finalised in such a way that the obstructions to visibility do not cause restrictions to the sight distance requirements.

The details of these geometric design factors are given in Chapter 4.

(d) *Economy* : The alignment finalised based on the above factors should also be economical. In working out the economics, the initial cost the cost, of maintenance and vehicle operation should be taken into account. The initial cost of construction can be decreased if high embankments and deep cuttings are avoided and the alignment is chosen in a manner to balance the cutting and filling.

(e) *Other Considerations* : Various other factors which may govern the alignment are drainage considerations, hydrological factors, political considerations and monotony. The vertical alignment is often guided by drainage considerations. The subsurface water level, seepage flow and high flood level are the factors to be kept in view.

A foreign territory coming across a straight alignment will necessitate deviation of alignment around the foreign land. At times the alignment is decided only on strategic considerations.

In a flat terrain it is possible to have a very long stretch of road, absolutely straight without horizontal curves. But straight road of very long stretch may be monotonous for driving. Hence after a few kilometers of straight road, it may be desirable to have a slight bend to break the monotony and keep the driver alert.

Special considerations while aligning roads on hilly areas

Stability : While aligning hill roads, special care should be taken to align the road along the side of the hill which is stable. A common problem in hill roads is that of land slides. The cutting and filling of earth to construct roads on hill-side causes steepening of existing slopes and affect its stability.

Drainage : Numerous hill-side drains should be provided for adequate drainage facility across the road. But the cross drainage structure being costly, attempts should be made to align the road in such a way where the number of cross drainage structures are minimum.

Geometric standard of hill roads : Different sets of geometric standards are followed in hill roads with reference to gradient, curves and speed and they consequently influence the sight distance, radius of curve and other related features. The route should enable the ruling gradient to be attained in most of the length, minimising steep gradients, hair pin bands and needless rise and fall.

Resisting length : The resisting length of a road may be calculated from the total work to be done to move the loads along the route taking the horizontal length, the actual difference in levels between the two stations and the sum of ineffective rise and fall in excess of *floating gradient*. In brief, the resisting length of the alignment should kept as low as possible. Thus the ineffective rise and executive fall should be kept minimum.

The detailed considerations on hill road alignment are discussed in chapter 12.

3.2 ENGINEERING SURVEYS FOR HIGHWAY LOCATIONS

Before a highway alignment is finalised in highway project, the engineering surveys are to be carried out. The surveys may be completed in four stages. The first three stages consider all possible alternate alignments keeping in view the various requirements of highway alignment as discussed in Art. 3.1.2. The fourth stage is meant for the detailed survey of the selected alignment.

The stages of the engineering surveys are

(a) Map study

(b) Reconnaissance

(c) Preliminary surveys

(d) Final location and detailed surveys.

3.2.1 Map Study

If the topographic map of the area is available, it is possible to suggest the likely routes of the road. In India topographic maps are available from the *Survey of India*, with 15 or 30 meter contour intervals. The main features like rivers, hills valleys etc. are also shown on these maps. By careful study of such maps, it is possible to have an idea of several possible alternate routes so that further details of these may be studied later at the site. The probable alignment can be located on the map from the following details available on the map.

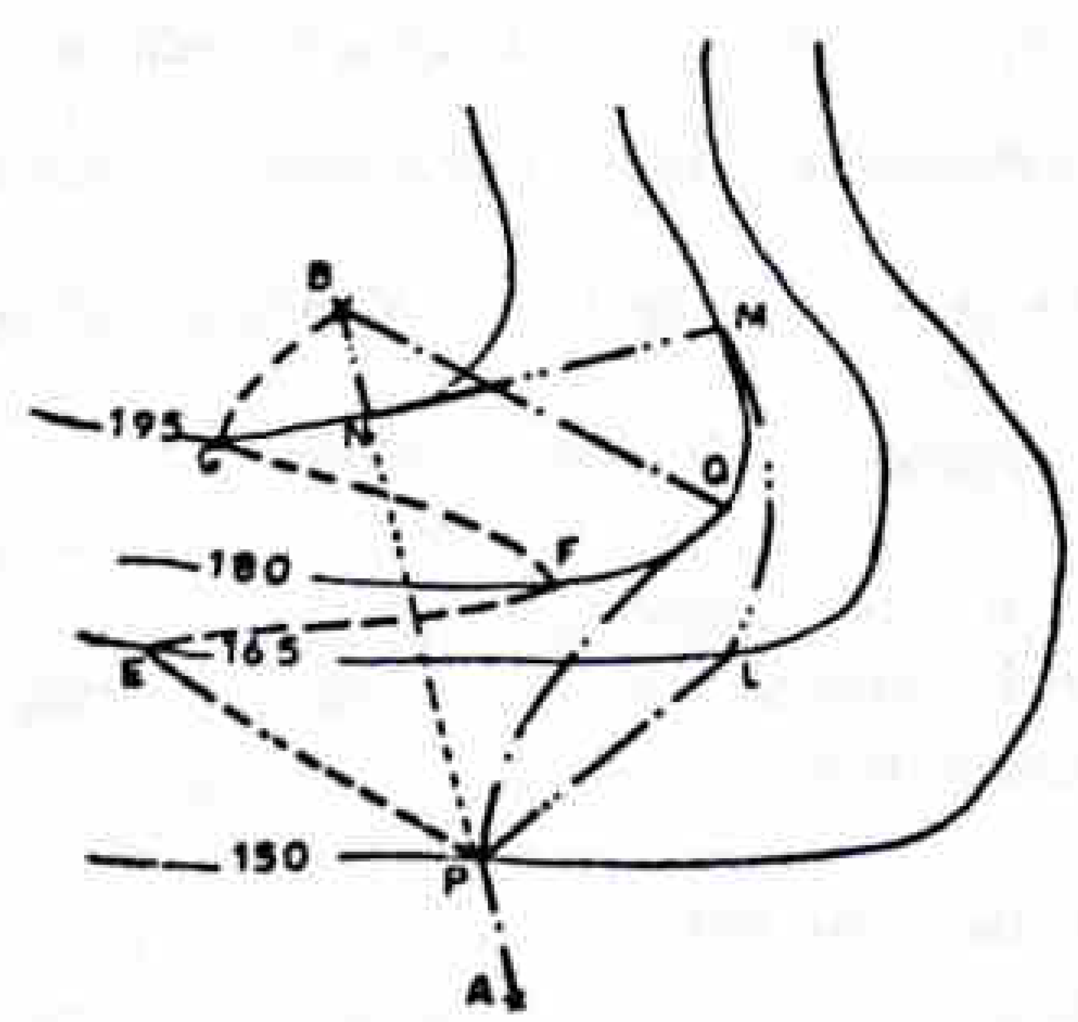
(a) Alignment avoiding valleys, ponds or lakes

(b) When the road has to cross a row of hills, possibility of crossing through a mountain pass

(c) Approximate location of bridge site for crossing rivers, avoiding bend of the river, if any.

(d) When a road is to be connected between two stations, one of the top and the other on the foot of the hill, then alternate routes can be suggested keeping in view the permissible gradient; say the ruling gradient. Refer Fig. 3.2. Suppose the scale of the contour map is known, then from the counter intervals it is possible to decide the length of road required between two consecutive contours, keeping the gradient within allowable limits. In this case, the contour interval is 15 metre and if the ruling gradient is fixed as 1 in 20, the road length between two consecutive contours has to be $15 \times 20 = 300$ meter. With the known scale of the map, the various possible alternate routes may be drawn by drawing arcs of the above (300 metre) length between the consecutive contour lines.

Let A and B be two stations to be connected by a road, see Fig. 3.2. AB is the shortest route (straight line) APQB is a steep route in which the gradient positively exceeds 1 in



AB - Shortest route
APQB - Steeper gradient
APLMNB - Flatter gradient
APEFGB - Flatter gradient (Alternate route)

Fig. 3.2 Alignment with allowable Gradients

20 as the distance between the contour interval is only about 200 metre (assuming the scale to be 1 cm = 150 metre). APLMNB is a route with an approximate slope of 1 in 20 whereas APEFGB is an alternate alignment with the same gradient.

Thus from the map study alternate routes can be suggested. It may also be possible from map study to drop a certain route in view of any unavoidable obstructions or undesirable ground, enroute. Map study thus gives a rough guidance of the routes to be further surveyed in the field.

3.2.2 Reconnaissance

The second stage of surveys for highway location is the reconnaissance to examine the general character of the area for deciding the most feasible routes for detailed studies. A field survey party may inspect a fairly broad stretch of land along the proposed alternative routes of the map in the field. Only very simple instrument like abney level, tangent clinometer, barometer etc. are used by the reconnaissance party to collect additional details rapidly (not accurately). All relevant details not available in the map are collected and noted down. Some of the details to be collected during reconnaissance are given below :

(i) Valleys, ponds, lakes, marshy land, ridge, hills, permanent structures and other obstructions along the route which are not available in the map.

(ii) Approximate values of gradient, length of gradients and radius of curves of alternate alignments.

(iii) Number and type of cross drainage structures, maximum flood level and natural ground water level along the probable routes.

(iv) Soil type along the routes from field identification tests and observation of geological features.

(v) Sources of construction materials, water and location of stone quarries.

(vi) When the road passes through hilly or mountainous terrain, additional data regarding the geological formation, type of rocks, dip of strata, seepage flow etc. may be observed so as to decide the stable and unstable sides of the hill for highway alignment.

A rapid reconnaissance of the area, especially when it is vast and the terrain is difficult, may be done by an aerial survey.

From the details collected during the reconnaissance, the alignment proposed after study may be altered or even changed completely. As a result of the reconnaissance a few alternate alignments may be chosen for further study based on practical considerations observed at the site.

3.2.3 Preliminary Survey

The main objectives of the preliminary survey are :

(i) To survey the various alternate alignments proposed after the reconnaissance and to collect all the necessary physical information and details of topography, drainage and soil.

(ii) To compare the different proposals in view of the requirements of a good alignment.

(iii) To estimate quantity of earth work materials and other construction aspects and to workout the cost of alternate proposals.

(iv) To finalise the best alignment from all considerations.

The preliminary survey is carried out to collect all the physical information which are necessary in connection with the proposed highway alignment. The preliminary survey may be carried out by any one of the following methods :

(a) Conventional approach, in which a survey party carries out surveys using the required field equipment, taking measurements, collecting topographical and other data and carrying out soil survey.

(b) Modern rapid approach, by serial survey taking the required aerial photographs and by photogrammetric methods and photo-interpretation techniques for obtaining the necessary topographic and other maps including details of soil and geology.

The procedure of the conventional methods of preliminary survey is given in following steps :

(i) *Primary traverse* : The first step in the preliminary survey is to establish the primary traverse, following the line recommended in the reconnaissance. For alternate alignments either secondary traverses or independent primary traverses may be necessary. As these traverses are *open traverses* no adjustment of errors is possible later, so the angles should be very accurately measured by the theodolite. The length of the centre line should be measured by using very good and accurate chaining methods or by tacheometry or by modern instruments.

(ii) *Topographical features* : After establishing the centre lines of preliminary survey, the topographical features are recorded. All geographical and other man made features along the transverse and for a certain width on either side are surveyed and plotted. The width to be surveyed is generally decided by the survey party, but the absolute minimum width is the land width of the proposed alignment.

(iii) *Levelling work* : Levelling work is also carried out side by side to give the centre line profiles and typical cross sections. The levelling work in the preliminary survey is kept to a minimum just sufficient to obtain the approximate earth work in the alternate alignments.

To draw contours of the strip of land to be surveyed, cross section levels should be taken at suitable intervals, generally 100 to 200 metre in plain terrain, upto 50 metre in rolling terrain and upto 30 metre in hilly terrain.

(iv) *Drainage studies and Hydrological data* : Drainage investigations and hydrological data are collected so as to estimate the type, number and approximate size of cross drainage structures. Also the vertical alignment of the highway, particularly the grade line is decided based on the hydrological and drainage data, such as HFL, ponded water level, depth of water table, amount of surface runoff, etc.

(v) *Soil survey* : Soil survey is an essential part of the preliminary survey as the suitability of the proposed location is to be finally decided based on the soil survey data. The soil survey conducted at this stage also helps in working out details of earth work, slopes, suitability of materials, subsoil and surface drainage requirements and pavement type and the approximate thickness requirements. All these details are required to make a comparative study of alternate proposals.

At this stage a detailed soil survey is not necessary. Post hole auger or any other suitable types of hand augers depending on the soil type, may be used to collect the soil sample up to a depth of 1 to 3 metre below the likely finished road level or the existing ground level, whichever is lower. When the road is expected to be constructed over an embankment, the depth of exploration should extend upto twice the height of embankment from the ground level. During the soil exploration if the ground water table is struck, the depth from the ground surface is also noted.

When the work has to be done rapidly, *geophysical method* of soil exploration are best suited as accuracy is not very important during the preliminary survey. The *electrical resistivity* method is commonly used in road projects. The method is based on the principle that the earth and rock materials may be identified by the different values of the resistance to flow of a direct current.

The soil samples collected during the field work are subjected to identification and classification test in the laboratory. *Soil profile* is obtained by drawing the longitudinal section along the proposed road alignment upto the depth of exploration. The types of soils encountered along the route upto the depth under consideration are marked on the soil profile either symbolically or by suitable colour coding.

(vi) *Material survey* : The survey for naturally occurring materials like stone aggregates, soft aggregates, etc. and identification of suitable quarries should be made. Also availability of manufactured materials like cement, lime, brick, etc. and their locations may be ascertained.

(vii) *Traffic survey* : Traffic surveys conducted in the region form the basis for deciding the number of traffic lanes and roadway width, pavement design and economic analysis of highway project. Traffic volume counts of the classified vehicles are to be carried out on all the existing roads in the region, preferably for 24 hours per day for seven days. Origin and destination surveys are very useful for deciding the alignment of the roads. This study may be carried out on a suitable sample of vehicle users or drivers. In addition the required traffic data may also be collected so that the traffic forecast could be made for 10 to 20 year periods.

(viii) *Determination of final centre line* : After completing the preliminary surveys and conducting the comparative studies of alternative alignments the final centre line of the road is to be decided in the office before the final location survey. For this, the preliminary survey maps consisting of contour plans, longitudinal profile and cross sections of the alternate alignments should be prepared and carefully studied to decide the best alignment satisfying engineering, aesthetic and economical requirements. After selecting the final alignment, the grade lines are drawn and the geometric elements of the horizontal and vertical alignments of the road are designed.

Aerial photographic surveys are very much suited for preliminary surveys, especially when the distance and area to be covered are vast. The survey may be divided into the following steps :

(a) Taking aerial photographs of the strips of land to be surveyed with the required longitudinal and lateral overlaps. Vertical photographs are necessary for the preparation of mossaics.

(b) The photographs are examined under stereoscopes and control points are selected for establishing the traverses of the alternate proposals. The control points are located on the maps.

(c) Using stereo-pair observations, the spot levels and subsequently contour lines may be obtained. Also from the stereo pairs the topographical details may be noted down on the maps.

(d) Photo-interpretation methods are used to assess the geological features, soil conditions, drainage requirements etc.

3.2.4 Final Location and Detailed Survey

The alignment finalised at the design office after the preliminary survey is to be first located on the field by establishing the centre line. Next detailed survey should be carried out for collecting the information necessary for the preparation of plans and construction details for the highway project.

Location

The centre line of the road finalised in the drawings is to be translated on the ground during the location survey. This is done using a transit theodolite and by staking of the centre line. The location of the centre line should follow, as closely as practicable, the alignment finalised after the preliminary surveys. Major and minor control points are established on the ground and centre pegs are driven, checking the geometric design requirements. However modifications in the final location may be made in the field, if found essential. The centre line stakes are driven at suitable intervals, say at 50 metre intervals in plain and rolling terrains and at 20 metre in hilly terrain.

Detailed survey

Temporary bench marks are fixed at intervals of about 250 metre and at all drainage and under pass structures. Levels along the final centre line should be taken at all staked points. Levelling work is of great importance as the vertical alignment, earth work calculations and drainage details are to be worked out from the level notes. The cross section levels are taken upto the desired width, at intervals of 50 to 100 metre in plain terrain, 50 to 75 metre in rolling terrain, 50 metre in built-up areas and 20 metre in hilly terrain. The cross sections may be taken at closer intervals at horizontal curves and where there is abrupt change in cross slopes. All river crossing, valleys etc. should be surveyed in detail upto considerably distances on either side.

All topographical details are noted down and also plotted using conventional signs. Adequate hydrological details are also collected and recorded.

A detailed soil survey is carried out to enable drawing of the soil profile. The depth upto which soil sampling is to be done may be 1.5 to 3.0 metre below the ground line or finished grade line of the road whichever is lower. However in case of high embankments, the depth should be upto twice the height of the finished embankment. The spacing of auger borings very much depends upon the soil type and its variations. CBR value of soils along the alignment may be determined for designing the pavement.

The data during the detailed survey should be elaborate and complete for preparing detailed plans, design and estimates of the project.

3.3 DRAWINGS AND REPORT

3.3.1 Drawings

The following drawings are usually prepared in a highway project :

(i) Key map

(ii) Index map

(iii) Preliminary survey plans

(iv) Detailed plan and longitudinal section

(v) Detailed cross-section

(vi) Land acquisition plans

(vii) Drawings of cross drainage and other retaining structures

(viii) Drawings of road intersections

(ix) Land plans showing quarries etc.

Key map should show the proposed and existing roads, and important places to be connected. The size of the plan generally should not exceed 22 × 20 cm. The scale of the map is chosen suitably depending upon the length of road.

Index map should show the general topography of the area. The details are symbolically represented. The index map should also be of suitable scale, the size being 32 × 20 cm.

Preliminary survey plans showing details of the various alternate alignments and all informations collected should be normally drawn to scale of 10 cm = 1 km to 25 cm = 1 km.

Detailed plans show the ground plan with alignment and the boundaries, contours at intervals of 1 to 2 metre in plain country a scale of 1/2400 and in close country, a scale of 1/1200 may be adopted for detailed plans. The size of the drawing may be A-2 size or 60 × 42 cm approximately.

Longitudinal sections should be drawn to the same horizontal scale of the ground as in detailed plan. Vertical scale may be enlarged 10 times of the longitudinal scale. The longitudinal section should show the details such as datum line, existing ground surface, vertical profile of the proposed road and position of drainage crossings.

Detailed cross sections are generally drawn to natural scale of 1 cm = 2.0 to 2.5 m. In Cross section should be drawn every 100 m or where there are abrupt changes in level. The cross section hill roads the cross sections should be drawn at closer intervals. The cross section drawings should extend at least up to the proposed right of way. The cross section number, the reduced distances and the area of filling and/or cutting should be shown on cross section drawings.

Land acquisition plans and schedules are usually prepared from the survey drawings for land acquisition details. These plans show all general details such as buildings, wells, nature of gradients and other details required for assessing the values. The scale adopted may be 1 cm = 40 m or less.

Detailed design for cross drainage and masonry structures are usually drawn to scale of 1 cm = 1 m. For details of any complicated portion of the structure enlarged scales up to 8 cm = 1 m or upto half full size may be employed. However the size of drawing should not exceed the standard size. Cross sections of streams should be to a scale of not less than 1 cm = 10 m.

Drawings of road intersections should be prepared showing all details of pavement, shoulders, islands etc. to scale.

Land plans for quarries. Where quarries for construction materials are to be acquired for new projects, separate land plans should be prepared. The size of these maps and scales may be similar to those suggested under land acquisition.

3.3.2 Estimates

The project estimates should consist of general abstract of cost and detailed estimates for each major head. If the project work is proposed to be executed in stages, the estimate should be prepared for each stage separately.

3.3.3 Project Report

The project report forms an important part of the project document. It should contain information such as

(i) general details of the project and its importance

(ii) feature of the road including selection of the route, alignment, traffic, etc.

(iii) road design and specifications

(iv) drainage facilities and cross drainage structures

(v) materials, labour and equipment

(vi) rates

(vii) construction programming and

(viii) other miscellaneous items like diversion roads, traffic control, road side amenities, rest houses, etc.

3.4 HIGHWAY PROJECT

3.4.1 General

In a new highway project, the engineer has to plan, design and construct either a network of new roads or a road link. There are also projects requiring re-design and re-alignment of existing roads of upgrading the geometric design standards.

Once a highway is constructed, development takes place along the adjoining land and subsequent changes in alignment or improvements in geometric standards become very difficult. A badly aligned highway is not only a source of potential traffic hazard, but also causes a considerable increase in transportation cost and strain on the drivers and the passengers. Therefore, proper investigation and planning are most important in a road project, keeping in view the present day needs as well as the future developments of the region.

3.4.2 New Highway Project

The new highway project work may be divided into the following stages :

(i) Selection of route, finalisation of highway alignment and geometric design details

(ii) Collection of materials and testing of subgrade soil and other construction materials mix design of pavement materials and design details of pavement layers.

(iii) Construction stages including quality control.

Route selection

The selection of route is made keeping in view the requirements of alignment and the geological, topographical and other features of the locality as explained in Art.3.2. However special care should be taken as regards the geometric design standards of the road for possible upgrading of speed standards in future, without being necessary to re-align the road. After the alignment if finalised, the plans and working drawings are prepared. The geometric design requirements of highways have been given in Chapter 4 and the details and requirements of hill roads are given in Chapter 12.

Materials and design

The soil samples collected from the selected route during the soil surveys are tested in the laboratory in order to design the pavement thickness required and the design of embankment and cut slopes. The basic construction materials such as selected soil, aggregates etc. are collected from the nearest borrow pits and quarries and stacked along the roads alignment after subjecting these materials to the specified laboratory tests. In order to design the mixes for the pavement component layers and to specify quality control test values during road construction, mix design tests are carried out in the laboratory.

The possibility of using low-cost construction material like soil-aggregate mixes, soft aggregates, stabilized soil and pozzolanic concrete mixes, in the sub-base or base course layers of pavement should be fully explored. When high quality pavement materials like bituminous mixes or cement concrete are to be used in the surface course, the mix design specification and construction control tests should be strictly followed. The pavement thickness is designed based on anticipated traffic, stability and drainage conditions of the subgrade and the type and thickness of pavement layers chosen for the construction. In India, the *CBR method* has been recommended by the Indian Roads Congress for designing the thickness of flexible pavements. Recommended procedure for the design of cement concrete payment has also been specified by the Indian Roads Congress. (Please see Chapter 7 for details).

Construction

The construction of the road may be divided into two stages, viz. : (i) earth work (ii) pavement construction. The earth work consists of excavation and construction of the embankments. During the excavation for highway cuts, the earth slopes, their protection and construction of drainage network are taken care of. Highway embankments may be best constructed by rolled-fill method by compacting the soil in layers under controlled moisture and density using suitable rollers. In the case of high embankments, the stability of the embankment foundation and slopes and the possible settlement of the embankment with time have to be investigated.

The pavement construction is subsequently taken up starting with the preparation of subgrade and the construction of sub-base, base and surface courses of the pavement.

Steps in a new project work

The various steps in a new highway project may be summarised as given below :

(i) *Map Study* : with the help of available topographic maps of the area.

(ii) *Reconnaissance Survey* : a general idea of a topography and other features, field identification of soils and survey of construction materials, by an on-the-spot inspection of the site.

(iii) *Preliminary Survey* : Topographic details and soil survey along alternate alignments, consideration of geometric design and other requirements of alignment, preparation of plans and comparison of alternate routes; economic analysis and selection of final alignment. Typical plan, longitudinal section and cross section drawing for the new alignment are shown in Fig. 3.3 a & b.

(iv) *Location of Final Alignment* : Transfer of the alignment from the drawings to the ground by driving pegs along the centre line of finally chosen alignment; setting out geometric design elements by location of tangent points, apex points, circular and transition curves, elevation of centre line and superelevation details.

(v) *Detailed Survey* : Survey of the highway construction work for the preparation of longitudinal and cross sections, computations of earth work quantities and other construction material; and checking details of geometric design elements.

(vi) *Materials Survey* : Survey of construction materials, their collection and testing.

(vii) *Design* : Design details of embankment and cut slopes, foundation of embankments and bridges, and pavement layers.

(viii) *Earth Work* : Excavations for highway cutting and drainage system, construction of embankments.

(ix) *Pavement Construction* : Preparation of subgrade, construction of sub-base and surface courses.

(x) *Construction Controls* : Quality control tests during different stages of constructions and check for finished road surface such as unevenness, camber, superelevation and extra widening of pavements at curves.

3.4.3 Re-alignment Project

Most of the present highways in India have been upgraded in stages, from the existing local roads of the pre-automobile era. As these roads were then meant for slow traffic, they are found deficient in the geometric design elements for the present day automobile traffic. There are several stretches of National Highways in the country having single lane of carriage way, narrow bridges and culverts and many locations with sharp horizontal curves and avoidable zig-zags, steep gradients and inadequate sight distances. These defects are to be rectified as early as possible atleast in roads of greater importance like National and State Highways. It will be worth while to adopt more liberal values of geometric design parameters than the ruling minimum values specified, where the conditions are favourable and the costs involved are not excessive. In such cases, it

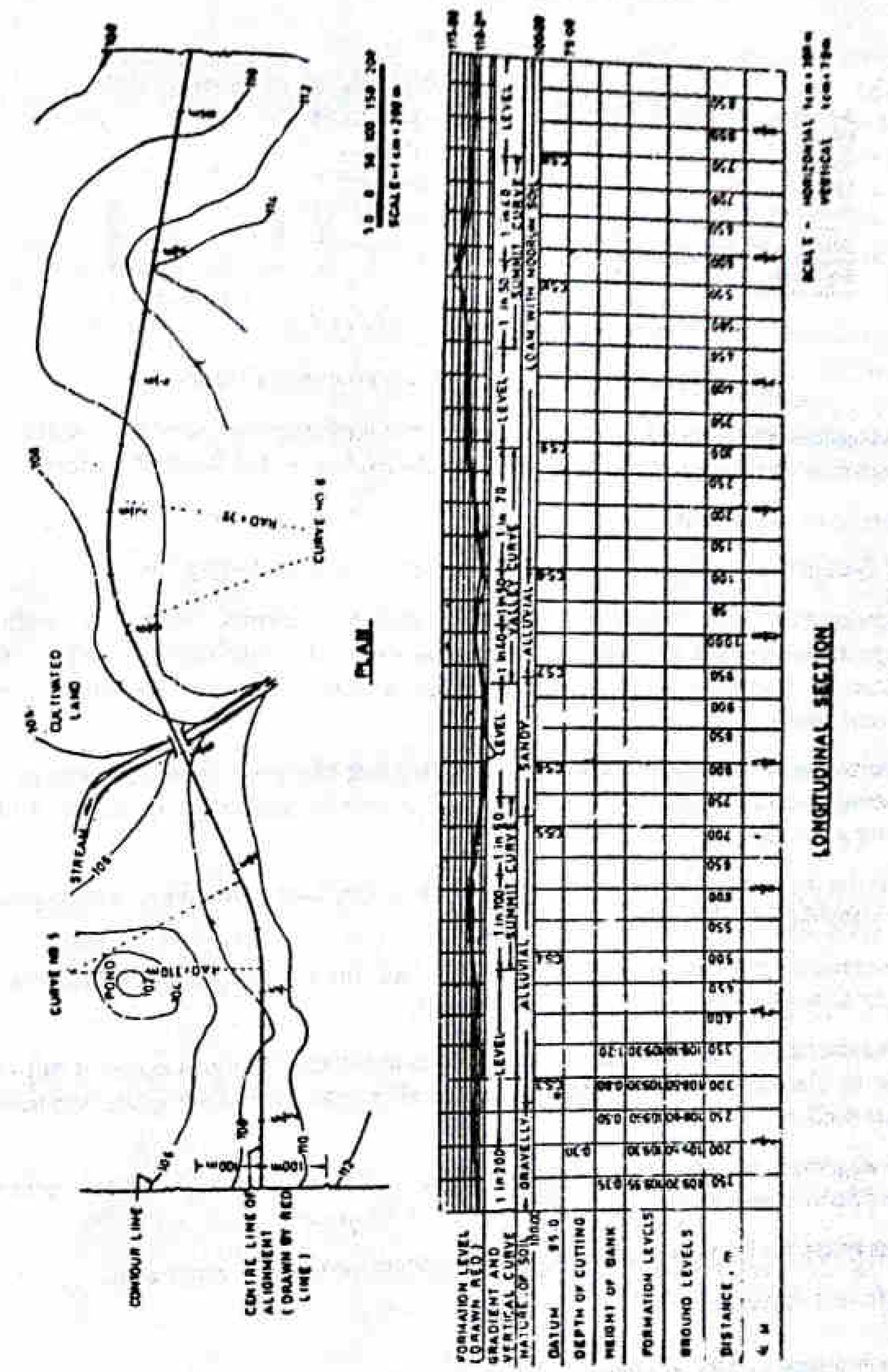


Fig.3.3 (a) Typical Drawing for a Highway Project

would be possible to upgrade the highways if necessary in future by increasing the width standards only, but without the necessity of re-aligning the road. However, in constrained situations and in difficult terrain, it may not always be economical to improve the existing highway geometric to the recommended design standards.

It has been decided as a policy that National Highways should as far as possible be able to fully cater to the traffic moving at design speed, fulfilling the comfort and safety requirements, both for the present and future traffic needs. To achieve this objective, it is necessary to plan improvements in the geometrics of roads wherever deficient, to the

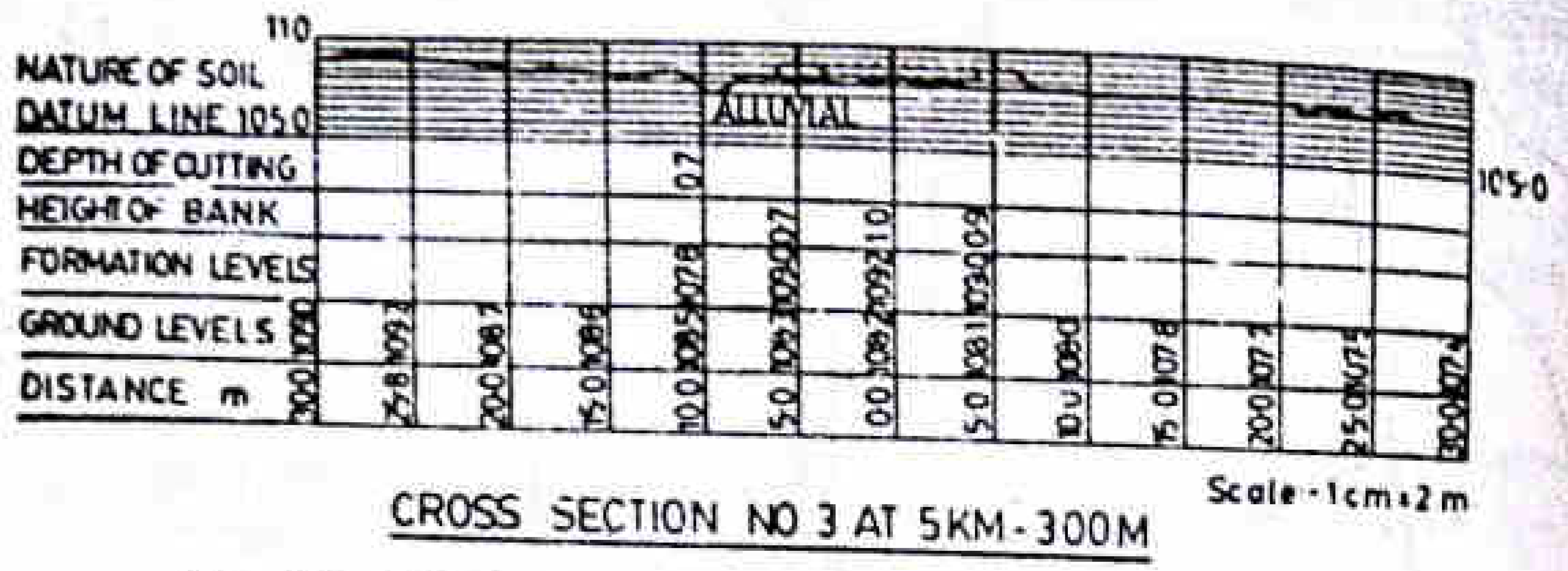


Fig. 3.3 (b) Typical Drawing for a Highway Project

extent economically practicable along with the other improvements such as re-surfacing the pavements, construction of overlay of raising the road above the flood water level.

Necessity of re-alignment

The re-alignment of existing roads may be necessary in the following cases :

(i) Improvement of horizontal alignment design elements, such as radius, superelevation, transition curve, clearance on inner side of the curve of shifting the curve to provide adequate sight distance, elimination of reverse curves and undesirable zig-zags, etc.

(ii) Improvement of vertical alignment design elements like steep gradients, changes in summit curves to increase sight distance, correction of undesirable undulations like humps and dips, etc.

(iii) Raising the level of a portion of a road which is subjected to flooding, submergence or water-logging during monsoons.

(iv) Re-construction of weak and narrow bridges and culverts and changes in water-way at locations slightly away from the existing site.

(v) Construction of over-bridges or under-bridges at suitable locations across a railway line in place of level crossing or across another road to provide grade separated inter-section.

(vi) Re-alignment required due to a portion of the road being submerged under water at the reservoir area on account of construction of a new dam.

(vii) Construction of a bypass to avoid the road running through a town or city.

(viii) Defence requirements.

General Principles of Re-alignment

(i) While improving the horizontal alignment of roads, improvement in sharp curves and zig-zags should be done after considering the whole alignment and not on piece meal basis. The improvement of transition curves would not generally be very costly and therefore the defects should be rectified where-ever necessary. The sight distance available generally gets increased when the horizontal alignment is improved; otherwise the set back distance may be increased at horizontal curves by removing or shifting the obstruction from the inner side of the curve, upto the desired extent.

(ii) While improving the vertical alignment, attempts should be made to provide overtaking sight distance at summit curves. However, if this is not possible, atleast the stopping sight distance should be available for the design speed at all locations of the road. The corrections of minor undulations such as humps and dips may not involve high cost and so it is desirable to provide suitable vertical transition curves for shock-free movement of vehicles travelling at the design speed. Valley curves may be checked for comfort condition and for visibility under the head lights of the vehicles during night driving.

(iii) The road stretches which remain submerged under water even for a short duration of the gear or those which are in water logged areas should be raised before strengthening or widening pavement section. The formation level should be raised such that the subgrade is at least 0.6 m above the *HFL*. Suitable measures should be adopted against water-logging and care should be taken to provide suitable drainage facilities, including the cross drainage works.

(iv) While reconstructing bridges of length greater than 60 m on sites other than the existing ones, separate surveys should be carried out for the selection of suitable sites. The selection of site for major bridges would be governed by the river training works, sub-soil conditions for foundation and hydraulic considerations. However, in small bridges the road alignment would essentially govern the bridge site selection.

(v) The deciding factor which is being considered for providing over-bridges or under bridges for a National Highway across a railway level crossing is the product of number of gate closures and the intensity of traffic on the highway in tonnes per day in the design year. When this product exceeds 50,000 or when the level crossing is within the shunting limits of a railway station, the grade separation is justified. The location is decided keeping in view the highway alignment, the topographic and other site conditions.

(vi) The necessity to provide alternate routes to bypass through-traffic is assessed from the origin and destination studies. If the bypassable traffic is more than the traffic terminating at the town or built-up area, then the bypass may be justified.

Steps in the Re-alignment Project

(i) Reconnaissance of the stretch of road to be re-aligned, study of the deficiencies and the possible changes in alignment.

(ii) Survey of existing road recording the topographic features and all other existing features including drainage conditions along a strip of land on either side of the road. The width of the land to be surveyed depends on the amount of shifting anticipated when the road is re-aligned. The field work may be carried out using plane table and level or by tacheometry.

(iii) Observations of spot levels along the centre line of the road and cross section levels at suitable intervals to note the gradient, cross slope, superelevation etc. The cross section levels should be taken at closer intervals at horizontal and vertical curves and at cross drainage works.

(iv) Soil survey along the stretches of land through which the re-aligned road may possibly pass; preparation of typical soil profiles after testing the soil samples in the laboratory.

(v) Comparison of economics and considerations of feasibility of alternate proposals of re-alignment and special study of stretches which are difficult for the re-alignment.

(vi) Finalisation of the design features of re-aligned road stretches.

(vii) Preparation of drawings (Typical drawings showing plan, longitudinal section and cross section for a re-alignment project are shown in Fig. 3.4).

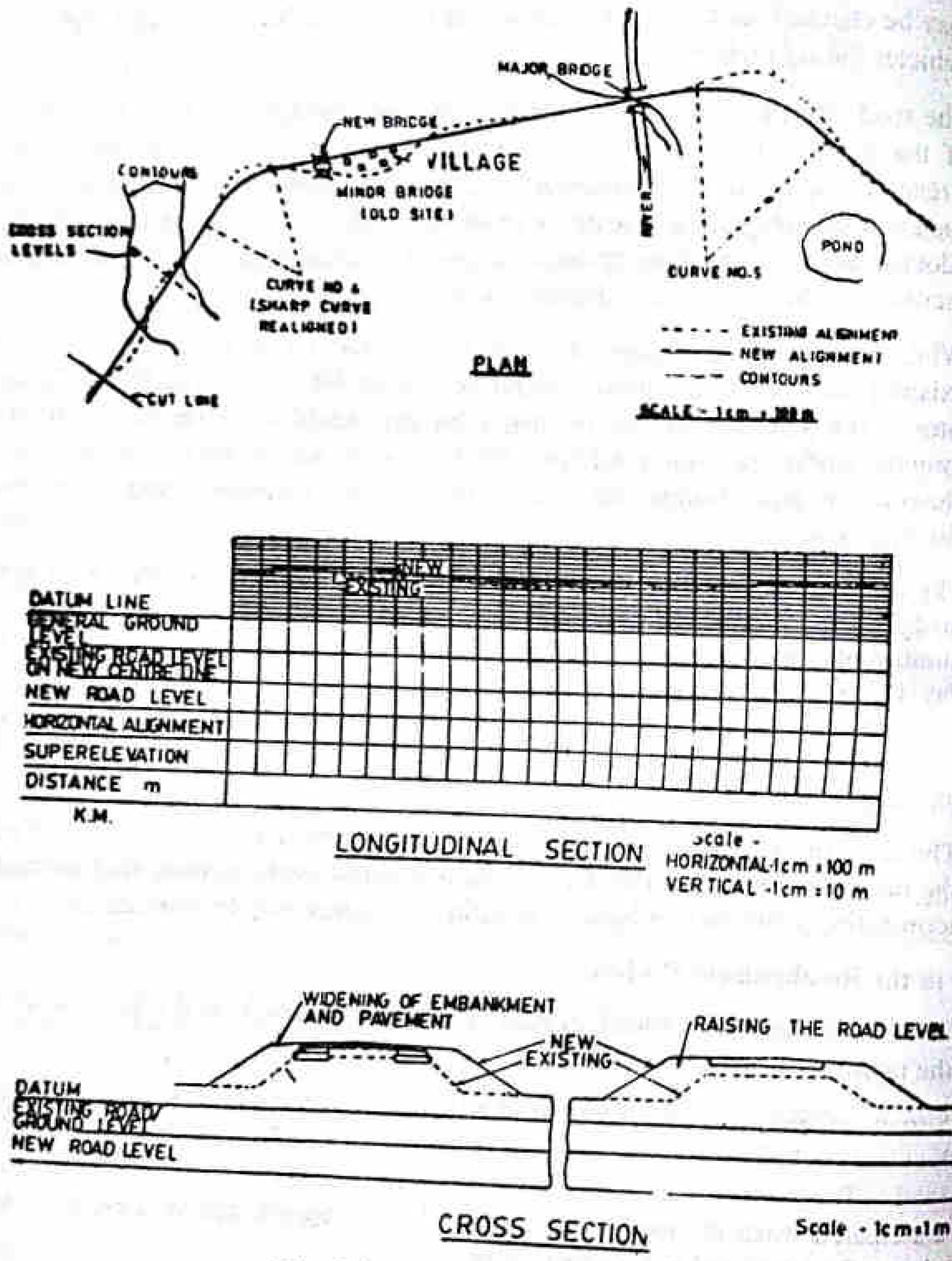


Fig. 3.4 Re-alignment Project

(viii) Marking out the centre line of re-aligned road while trying to utilise the existing road to the maximum extent possible.

(ix) Earth-work and preparation of subgrade of the re-alignment road stretches, setting out and construction of new bridges and culverts.

(x) Checking the geometric design elements of the newly aligned stretches of the road.

(xi) Design and construction of the new highway pavements.

Preparation of Drawings for Re-alignment Project

The drawings for the re-alignment project should show all the existing features of the road as well as all the proposed improvements. The following drawings would be needed :

(i) Plan showing existing road, proposed alignment, contours and all other features of importance.

(ii) Longitudinal section showing natural ground elevation, surface of the existing road and the grade line for the re-construction.

(iii) Cross section showing the existing roadway and new roadway drawn at 250 m intervals on straights, at the beginning and end of transition curves and at the middle of circular curves. Cross sections are drawn at 50 m interval where the new carriageway falls entirely outside the existing one.

A typical set of such drawings is shown in Fig. 3.4. The drawings along with a comprehensive report to justify the re-alignment project should be prepared.

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PROBLEMS

1. What are the various requirements of an ideal highway alignment. Discuss briefly.
2. Explain with sketches the various factors controlling the alignment of roads.
3. Explain obligatory points. With sketches, discuss how these control the alignment.
4. Discuss the special care to be taken while aligning hill roads.
5. Briefly explain the engineering surveys needed for locating a new highway.
6. What are the uses of map study in engineering surveys for highway location ?
7. What are the objects of reconnaissance in engineering surveys ? Discuss the scope of aerial survey for the purpose.
8. What are the various objectives of preliminary survey for highway alignment ? Enumerate the details to be collected and the various steps in the conventional method.

9. Discuss the scope of aerial surveys in preliminary survey for highway location. What are the steps to be followed ?
10. Explain how the final location and detailed survey of a highway are carried out.
11. Give the details of drawings to be prepared in highway project with the recommended scales and size of the drawings.
12. Explain briefly the various stages of work in a new Highway Project.
13. What are the conditions which necessitate taking up of a re-alignment project of the highway.
14. Discuss the general principles in the re-alignment of a highway and explain how the work is carried out.

Chapter 4

Highway Geometric Design

4.1 INTRODUCTION

4.1.1 Importance of Geometric Design

The geometric design of a highway deals with the dimensions and layout of visible features of the highway such as alignment, sight distances and intersections.

The geometrics of highway should be designed to provide optimum efficiency in traffic operations with maximum safety at reasonable cost. The designer may be exposed to either planning of new highway net work or improvement of existing highways to meet the requirements of the existing and the anticipated traffic.

It is possible to design and construct the pavement of a road in stages; but it is very expensive and rather difficult to improve the geometric elements of a road in stages at a later date. Therefore it is important to plan and design the geometric features of the road during the initial alignment itself taking into consideration the future growth of traffic flow and possibility of the road being upgraded to a higher category or to a higher design speed standard at a later stage.

Geometric design of highways deals with following elements :

(i) Cross section elements

(ii) Sight distance considerations

(iii) Horizontal alignment details

(iv) Vertical alignment details

(v) Intersection elements

Under cross section elements, the considerations for the width of pavement, formation and land, the surface characteristics and cross slope of pavement are included. The sight distance or clear distance visible ahead of a driver at horizontal and vertical curves and at intersections govern the safe movements of vehicles.

The change in the road directions are made possible by introducing horizontal curves. Super-elevation is provided by raising the outer edge of pavement to counteract the centrifugal force developed on a vehicle traversing a horizontal curve; extra pavement width is also provided on horizontal curves. In order to introduce the centrifugal force and the super-elevation gradually, transition curves are introduced between the straight and circular curves. The gradients and vertical curves are introduced in the vertical alignment of a highway. Design of road intersections with facilities for safe and efficient traffic movement needs adequate knowledge of traffic engineering.

Highway geometrics are greatly influenced by the topography, locality and traffic characteristics and the requirements of design speed. The factors which control the geometric design requirements are speed, road user and vehicular characteristics, design traffic, traffic capacity and benefit-cost considerations. However, speed is the factor which is important governing most of the geometric design elements of roads, as may be seen from the subsequent articles of this chapter.

4.1.2 Design Controls and Criteria

The geometric design of highways depends on several design factors. The important of these factors which control the geometric elements are :

(i) Design speed

(ii) Topography

(iii) Traffic factors

(iv) Design hourly volume and capacity

(v) Environmental and other factors.

(i) Design Speed

The design speed is the most important factor controlling the geometric design elements of highways. The design speed is decided taking into account the overall requirements of the highway. In India different speed standards have been assigned depending upon the importance or the class of the road such as National/State Highways, Major/Other District Roads and Village Roads. Further the design speed standards are modified depending upon the terrain or topography. Similarly urban roads have a different set of design speeds.

Design of almost every geometric design element of a road is dependent on the design speed. For example the requirements of the pavement surface characteristics, the cross section element of the road such as width and clearance requirements, the sight distance requirements, the horizontal alignment elements such as radius of curve super-elevation, transition curve length and the vertical alignments such as gradient, summit and valley curve lengths-all these depend mainly on the design speed of the road.

(ii) Topography

The topography or the terrain conditions influence the geometric design of highway significantly. The terrains are classified based on the general slope of the country across the alignment, as plain rolling, mountainous and steep terrains as given in Art. 4.4.2. The design standards specified for different classes of roads are different depending on the terrain classification. For example the design or ruling speed of NH and SH on plain terrain with general cross slope upto 10% is 100 kmph whereas the speed on rolling

terrain with general cross slope of 10 to 25% is 80 kmph and that on mountainous terrain with cross slope 25 to 60% is 50 kmph. As the speed standards affect every geometric design element, topography also affects the geometric design of roads. Further in hilly terrain, it is necessary to allow for steeper gradients and sharper horizontal curves due to the construction problems.

(iii) Traffic Factors

The factors associated with the traffic that affect geometric design of roads are the vehicular characteristics and human characteristics of road users. It is difficult to decide the design vehicle or the standard traffic lane under the mixed traffic flow condition prevalent especially on urban roads of developing countries. This is a complex problem. The different vehicle classes such as passenger cars, buses, trucks, motor cycles, etc. have different speed and acceleration characteristics, apart from having different dimensions and weights. However, it is often necessary to consider some standard vehicle as the design vehicle. The important human factors which affect traffic behaviour include the physical, mental and psychological characteristics of drivers and pedestrians.

(iv) Design Hourly Volume and Capacity

The traffic flow or volume keeps fluctuating with time, from a low value during off-peak hours to the highest value during the peak hour. It will be uneconomical to design the roadway facilities for the peak traffic flow or the highest hourly traffic volume. Therefore a reasonable value of traffic volume is decided for the design and this is called the design hourly volume. This value is to be determined from extensive traffic volume studies as discussed in Art. 5.2.3. The ratio of volume to capacity affects the level of service of the road.

(v) Environmental and Other Factors

The environmental factors such as aesthetics, landscaping, air pollution, noise pollution and other local conditions should be given due consideration in the design on road geometrics. Some of the arterial high speed highways and expressways are designed for higher speed standards and uninterrupted flow of vehicles by providing grade separated intersections and controlled access.

4.2 HIGHWAY CROSS SECTION ELEMENTS

4.2.1 Pavement Surface Characteristics

The pavement surface depends on the pavement type which is decided based on the availability of materials and funds, volume and composition of traffic, subgrade, and climatic conditions, construction facilities and cost considerations. The important surface characteristics of the pavement are the friction unevenness, light reflecting characteristics and drainage of surface water.

Friction

The friction between vehicle tyre and pavement surface is one of the factors determining the operating speed and distance requirements in stopping and accelerating the vehicles. When a vehicle negotiates a horizontal curve, the lateral friction developed counteracts the centrifugal force and thus governs the safe operating speed. Frictional force is an important factor in the acceleration and retardation abilities of vehicles. The coefficient of friction or the skid resistance offered by the pavement surface under various driving and surface conditions is important with reference to the safety. The maximum coefficient of friction comes into play only when the braking efficiency is high enough to partially arrest the rotation of the wheels on application of brakes, at low speeds.

Skid occurs when the slide without revolving or when the wheels partially revolve i.e., when the path travelled along the road surface is more than the circumferential movements of the wheels due to their rotation. When the brakes are applied, the wheels are locked partially or fully, and if the vehicle moves forward, the longitudinal skidding takes place which may vary from 0 to 100 percent.

While a vehicle negotiates a horizontal curve, if the centrifugal force is greater than the counteracting forces (i.e. lateral friction and component of gravity due to super-elevation) lateral skidding takes place. The lateral skid is considered dangerous as the vehicle goes out of control leading to an accident. The maximum lateral skid coefficient is generally equal to or slightly higher than the forward skid coefficient in braking tests.

Slip occurs when a wheel revolves more than the corresponding longitudinal movement along the roads. Slipping usually occurs in the driving wheel of a vehicle when the vehicle rapidly accelerates from stationary position or from slow speed on pavement surface which is either slippery and wet or when the road surface is loose with mud.

Factors affecting friction or skid resistance

The maximum friction offered by pavement surface or the skid resistance depends upon the following factors :

(i) Type of pavement surface namely, cement concrete bituminous, WBM, earth surface etc.

(ii) Macro-texture of the pavement surface or its relative roughness.

(iii) Condition of pavement namely, wet or dry, smoothened or rough, oil spilled, mud or dry sand on pavement.

(iv) Type and condition of tyre i.e. new with good treads or smoothened and worn out tyre.

(v) Speed of vehicle

(vi) Extent of brake application or brake efficiency

(vii) Load and tyre pressure

(viii) Temperature of tyre and pavement, and

(ix) Type of skid, if any

The type of aggregate used and the mix design of pavement surface course affect the skid resistance of the pavements, particularly in the case of old or worn out pavements.

The coefficient of friction reduces considerably when the pavement surface is smooth or wet. The coefficient of friction also decreases slightly with increase in temperature, tyre pressure and load. Smooth and worn out tyres offer higher friction factors on dry pavement then new tyres with treads because of large areas of contact. But on wet pavements new tyres with good treads give higher friction factors than worn out tyres.

This is because the lubricating effect of water on the wet pavement is reduced as the water entrapped between the tyre and pavement escapes into the treads of the tyre. Hence new tyres are more dependable than smooth ones in adverse surface and other conditions prone to skidding, such as wet pavements. The minimum anticipated value of coefficient of friction under worst possible pavement condition is generally taken for design purposes. The friction coefficient decreases with skid speed, which in turn depends on the speed of vehicle and brake efficiency.

For the calculation of stopping distance, the longitudinal friction coefficient values of 0.35 to 0.40 have been recommended by the Indian Roads Congress, depending upon speed. See Art. 4.3.2. These values have been suggested keeping in view the minimum coefficient of friction in the longitudinal direction on wet pavements and after allowing a suitable factor of safety. Further when a longitudinal friction coefficient of 0.40 is allowed for stopping the vehicle, the resultant retardation is 3.93 m/sec^2 which is not too uncomfortable to the passengers.

In the case of horizontal curve design, the Indian Roads Congress has recommended the lateral coefficient of friction of 0.15. This low value of transverse skid resistance has been suggested considering the worst possible surface condition such as mud on pavement surface at horizontal curve with super-elevation, during the rains, as it is essential to prevent possible lateral skid, even under adverse pavement conditions.

Pavement unevenness

Higher operating speeds are possible on even pavement surfaces with less undulations than on uneven and poor surfaces. Pavement surface should hence be maintained with minimum possible unevenness so that the desired speed can be maintained in conformity with other geometric standards. Pavement unevenness also affects vehicle operation cost, comfort and safety. Fuel consumption and wear and tear of tyres and other moving parts increases with increase in pavement unevenness. Loose road surfaces increase the tractive resistance and hence causes increase in fuel consumption. Uneven surfaces also increase fatigue and accidents.

The pavement surface condition is commonly measured by using an equipment called Bump Integrator, in terms of unevenness index, which is the cumulative measure of vertical undulations of the pavement surface recorded per unit horizontal length of the road. For example, unevenness index may be measured in cm per km. It has been shown from the tests that it is desirable to keep the unevenness index low, and preferably less than 150 cm/km for good pavement surfaces of high speed highways. A value of 250 cm/km is satisfactory upto a speed of about 100 kmph. Value more than 350 cm/km is considered very uncomfortable even at speed of 50 kmph. (See Art. 10.4.2 for evaluation of pavement surface condition). Pavement undulations are also some times measured using a straight edge in terms of the extent of number of depression or ruts along and across the pavement.

An Unevenness Indicator has been designed and patented by the Central Road Research Institute, New Delhi. This equipment is useful to indicate unevenness values from 3 to 20 mm.

The unevenness or undulations on pavement surface may be caused by various factors, such as : (i) inadequate or improper compaction of the fill, subgrade and pavement layers (ii) un-scientific construction practices including the use of boulder stones and bricks as soling course over loose subgrade soil (iii) use of inferior pavement materials (iv) improper surface and subsurface drainage (v) use of improper construction machinery (vi) poor maintenance practices and (vii) localized failures due to combination of causes.

Light reflecting characteristics

Night visibility very much depends upon the light reflecting characteristics of the pavement surface. The glare caused by the reflection of head lights is considerably high on wet pavement surface than on the dry pavement. Light colored or white pavement surface give good visibility at night particularly during rains, and they produces glare and eye strain during bright sunlight. Black top pavement surface on the other hand provides very poor visibility at nights, especially when the surface is wet.

4.2.2 Cross Slope or Camber

Cross slope or *chamber* is the slope provided to the road surface in the transverse direction to drain off the rain water from the road surface. Drainage and quick disposal of water from the pavement surface by providing cross slope is considered important because of two reasons :

(i) To prevent the entry of surface water into the subgrade soil through pavement; the stability, surface condition and the life of the pavement get adversely affected if the water enters in the subgrade and the soil gets soaked.

(ii) To prevent the entry of water into the bituminous pavement layers, as continued contact with water causes stripping of bitumen from the aggregates and results in deterioration of the pavement layer.

(iii) To remove the rain water from the pavement surface as quickly as possible and to allow the pavement to get dry soon after the rain; the skid resistance of the pavement gets considerably decreased under wet condition, rendering it slippery and unsafe for vehicle operation at high speeds.

Usually the camber is provided on the straight roads by raising the center of the carriageway with respect to the edges, forming a crown or highest point on the center line. At horizontal curves with super-elevation, the surface drainage is effected by raising the outer edge of pavement with respect to the inner edge while providing the desired super-elevation. The rate of camber or cross slope is usually designated by 1 in n which means that the transverse slope is in ratio 1 vertical to n horizontal. Camber is also expressed as a percentage. If the camber is x%, the cross slope is x in 100.

The required camber of a pavement depends on :

(i) the type of pavement surface, and

(ii) the amount of rainfall

A flat camber of 1.7 to 2.0% is sufficient on relatively impervious pavement surface like cement concrete or bituminous concrete. In pervious surface like water bound macadam or earth road which may allow surface water to get into the subgrade soil, steeper cross slope is required. Steeper camber are also provided in areas of heavy rainfall.

The minimum camber needed to drain off surface water may be adopted keeping in view the type of pavement surface and the amount of rainfall in the locality. Too steep cross slope is not desirable because of the following reasons :

(i) Transverse tilt of vehicles causes uncomfortable side thrust and a drag on the steering of automobiles. Also the thrust on the wheels along the pavement edges is more causing unequal wear of the tyres as well as road surface.

(ii) Discomfort causing throw of vehicle when crossing the crown during overtaking operations.

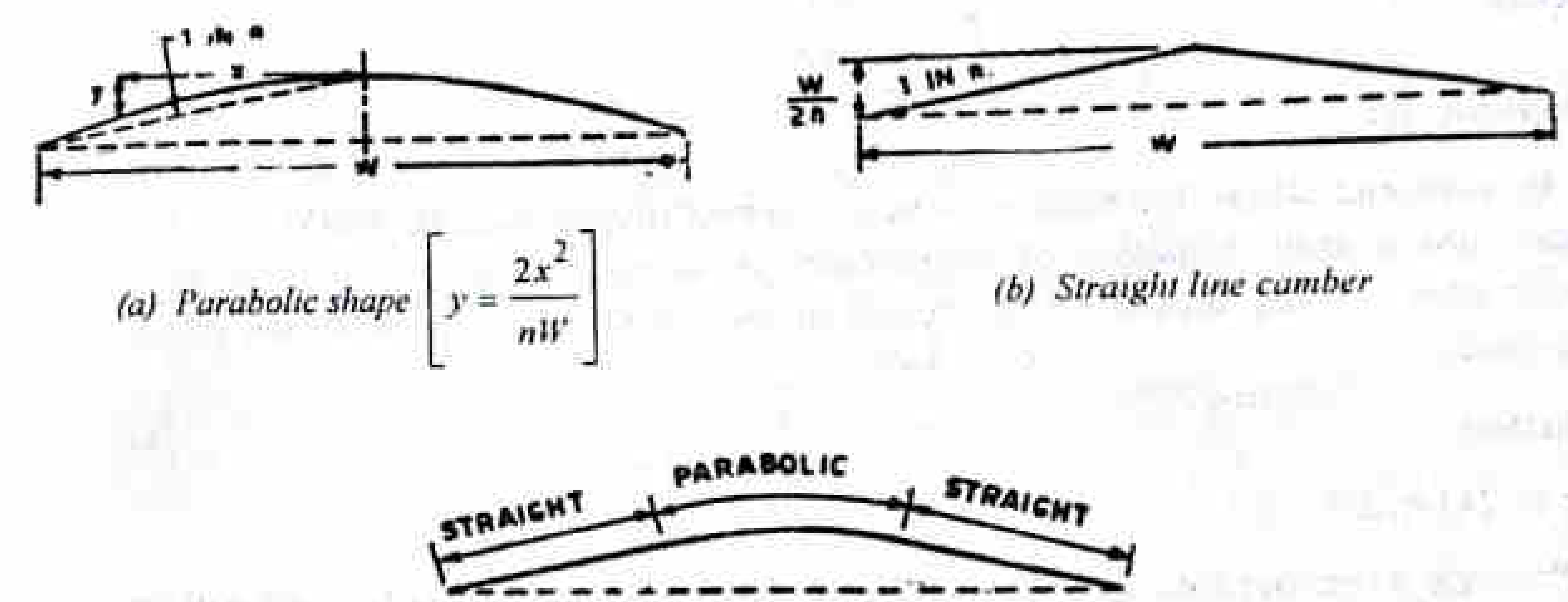
(iii) Problems of toppling over of highly laden bullock carts and trucks.

(iv) Formation of cross ruts due to rapid flow of water.

(v) Tendency of most of the vehicles to travel along the center line

Shape of cross slope

The camber is given a parabolic elliptic or straight line shape in the cross section. Parabolic or elliptic shape is given so that the profile is flat at the middle and steeper towards the edges, which is preferred by fast moving vehicles as they have to frequently cross the crown line during overtaking operation on a two lane highway. See Fig. 4.1a.



(a) *Parabolic shape* $\left[y = \frac{2x^2}{nW} \right]$ (b) *Straight line camber*

(c) *Combination of straight and parabolic shape*

Note : vertical scale are enlarged in the above sketches

Fig. 4.1 Shapes of Cross Slope

When very flat cross slope is provided as in cement concrete pavements, straight line shape of camber may be provided as shown in Fig. 4.1b. Steel tyred wheels of animal drawn vehicles can cause considerable damage to the pavement surface due to high stresses. The wheel does not have full contact increasing further the contact stress under these steel tyred wheels when the vehicle travels along the center of the pavement with straight camber. Some times a combined camber with parabolic central portion and straight line camber at the edges as shown in Fig. 4.1 c is preferred.

The values of camber recommended by the IRC for different types of road surfaces are given in table 4.1. A range of values are given with a view that in localities with lower rainfall, a flatter chamber and in places with high rainfall, a steeper camber can be adopted.

Table 4.1 Recommended values of camber for different types of road surfaces

Sl. No.	Types of road surface	Range of camber in areas of rainfall range		
		Heavy	to	Light
1.	Cement concrete and high type bituminous surface	1 in 50 (2.0 %)	to	1 in 60 (1.7 %)
2.	Thin bituminous surface	1 in 40 (2.5 %)	to	1 in 50 (2.0 %)
3.	Water bound macadam, and gravel pavement	1 in 33 (3.0 %)	to	1 in 40 (2.5 %)
4.	Earth	1 in 25 (4.0 %)	to	1 in 33 (3.0 %)

The cross slope for shoulders should be 0.5% steeper than the cross slope of adjoining pavement, subject to a minimum of 3.0% (and a maximum value of 5.0% for earth shoulders).

Providing camber in the field

For providing the desired amount and shape of camber, templates of camber boards are prepared with the specified camber. These are used to check the lateral profile of finished pavement during construction. Depending on the shape of the camber chosen, the camber board may be prepared. Forming a straight line camber is very simple. In the case of parabolic camber, the general equation $y = x^2/a$ may be adopted.

Here $a = nW/2$ for a pavement of width W and cross slope 1 in n.

Hence,
$$y = \frac{2x^2}{nW} \qquad (4.1)$$

Example 4.1

In a district where the rainfall is heavy, major district road of WBM pavement, 3.8 m wide, and a state highway of bituminous concrete pavement, 7.0 m wide are to be constructed. What should be the height of the crown with respect to the edges in these two cases ?

Solution

For WBM road

Provide a camber rate of 1 in 33 as the rainfall is heavy. Rise of crown with respect to edges

$$= \frac{3.8}{2} \times \frac{1}{33} = 0.058 \text{ m}$$

For bituminous concrete road

Provide a cross fall of 1 in 50.

Rise of crown with respect to the edges

$$= \frac{7}{2} \times \frac{1}{50} = 0.07 \text{ m}$$

4.2.3 Width of Pavement or Carriageway

The pavement or carriageway width depends on the width of traffic lane and number of lanes. The carriageway intended for one line of traffic movement may be called a traffic lane. The lane width is determined on the basis of the width of vehicle and the minimum side clearance which may be provided for the safety. When the side clearance is increased (up to a certain limit) there is an increase in operating speed of vehicles and hence an increase in capacity of the traffic lane. Keeping all these in view a width of 3.75 m is considered desirable for a road having single lane for vehicles of maximum width 2.44 m. For pavements having two or more lanes, width of 3.5 m per lane is considered sufficient.

The maximum width of vehicle as per IRC specifications is 2.44 m. For details refer Art. 5.2. If a single lane carriageway of width 3.8 m is provided, a side clearance of

0.68 m would be obtained as shown in Fig. 4.2a. In the case of two-lane pavement of width 0.7 m, a minimum clearance between two lanes of traffic would be 1.06 m for the widest vehicles on the road, as shown in Fig. 4.2b.

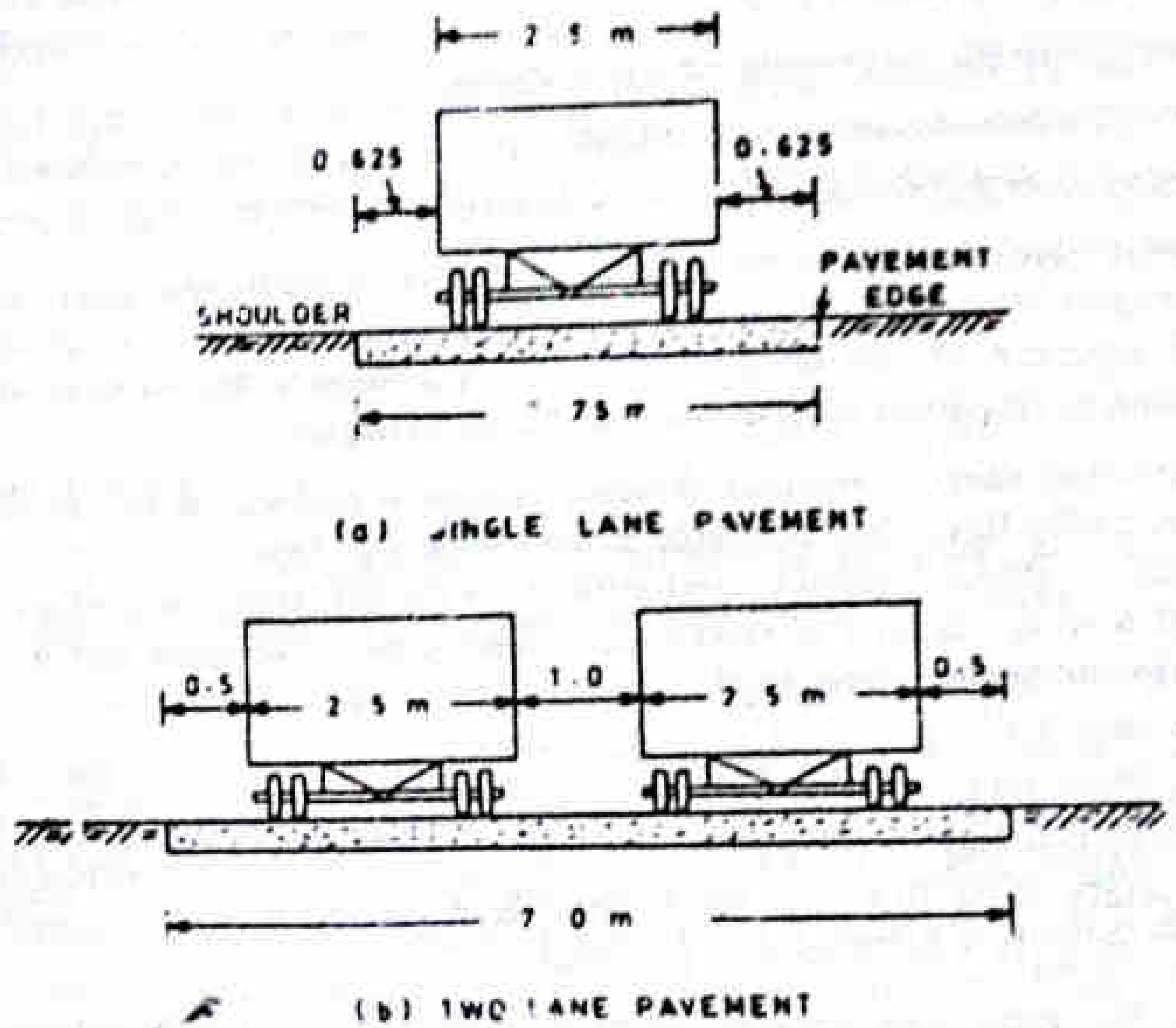


Fig. 4.2 Lateral Placement of Vehicles

The number of lanes required in a highway depends on the predicted traffic volume and the design traffic volume of each lane. The width of pavement is increased on horizontal curves as discussed in Art. 4.3.5.

In some highways, *traffic separators or medians* are provided between two sets of traffic lanes intended to divide the traffic moving in opposite directions. In such highways the road width depends on the pavement width (or the lane widths and number of lanes) and the width of traffic separators. The width of carriageway for various classes of roads standardised by Indian Roads Congress are given in Table 4.2.

Table 4.2 Width of Carriageway

	Class of road	Width of carriageway
(i)	Single lane	3.75 m
(ii)	Two lanes, without raised kerbs	7.0 m
(iii)	Two lanes, with raised kerbs	7.5 m
(iv)	Intermediate carriageway (except on important roads)	5.5 m
(v)	Multi-lane pavements	3.5 m per lane

Notes : (i) The width of single lane or village roads may be decreased to 3.0 m

(ii) On urban roads without kerbs the single lane width may be decreased to 3.5 m and in access roads to residential areas to 3.0 m

(iii) The minimum width recommended for kerbed urban road is 5.5 m to make allowance for a stalled vehicle.

Traffic separators of medians

The main function of traffic separator is to prevent head-on collision between vehicles moving in opposite directions on adjacent lanes.

The separators may also help to

(i) Channelize traffic into streams at intersections

(ii) shadow the crossing and turning traffic

(iii) segregate slow traffic and to protect pedestrians.

The traffic separators used may be in the form of pavement markings, physical dividers or area separators. Pavement marking is the simplest of all these. The mechanical separator should be designed in such a manner that even if wheels of a vehicle encroach, no part of vehicle body should be damaged.

Area separators may be medians, dividing islands or parkway strips, dividing the two directions of traffic flow. It is desirable to have wide area separators of 8 to 14 m width. But the width should be decided in conformity with the availability of land and its cost. A minimum of 6 m is required to reduce head light glare.' The glare can be reduced in narrower strips by planting trees or shrubs.

The IRC recommends a minimum desirable width of 5.0 m for medians of rural highways, which may be reduced to 3.0 m where land is restricted. On long bridges the width of the median may be reduced upto 1.2 to 1.5 m. The medians should normally be of uniform width on a particular road, but where change in width is unavoidable, a transition of 1 in 15 to 1 in 20 must be provided.

On urban highways with six lanes or more, medians should invariably be provided. The minimum recommended width of medians at intersections of urban roads are 1.2 m for pedestrian refuge, 4.0 to 7.5 m for protection of vehicles making right turn and 9.0 to 12 m for protection vehicles crossing at grade. The absolute minimum width of median in urban area is 1.2 m and desirable minimum is 5.0 m.

4.2.4 Kerbs

Kerb indicates the boundary between the pavement and shoulder; or sometimes islands or foot path or kerb parking space. It is desirable to provide kerbs on urban roads (See Fig. 4.3). There are a variety of kerb designs. Kerbs may be mainly divided into three groups based on their functions.

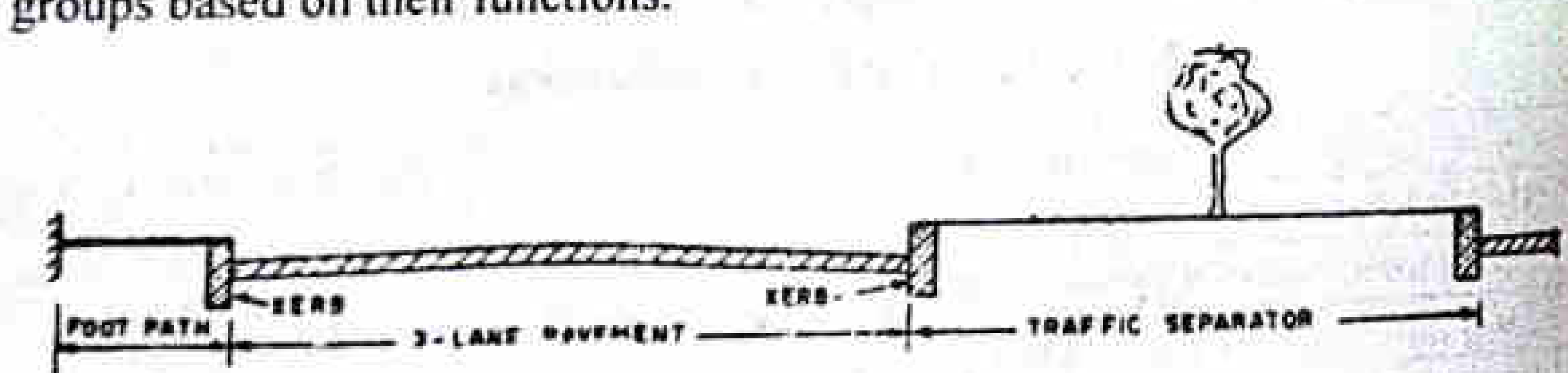


Fig. 4.3 Kerb and Traffic Separator

(i) *Low or mountable type kerbs* which though encourage traffic to remain in the through traffic lanes, yet allow the driver to enter the shoulder area with little difficulty. The height of this type of shoulder kerbs is about 10 cm above the pavement edge with a slope or batter to help vehicles climb the kerb easily. This type of kerb is provided at medians and channelization schemes and is also useful for longitudinal drainage system.

(ii) *Semi-barrier type kerb* is provided on the periphery of a roadway where the pedestrian traffic is high. This type of kerb has a height of about 15 cm above the pavement edge with a batter of 1 : 1 on the top 7.5 cm. This kerb prevents encroachment of parking vehicles, but at acute emergency it is possible to drive over this kerb with some difficulty.

(iii) Barrier type kerb is provided in built-up areas adjacent to foot paths with considerable pedestrian traffic. The height of kerb stone is about 20 cm above the pavement edge with a steep batter of 1.0 vertical 0.25 horizontal.

In rural roads submerged kerbs are sometimes provided at pavement edge between edge and shoulders. These kerbs provide lateral confinement and stability to the granular base course and flexible pavements.

4.2.5 Road Margins

The various elements included in the road margins are shoulder, parking lane, frontage road, driveway, cycle track, footpath, guard rail and embankment slope.

Shoulders are provided along the road edge to serve as an emergency lane for vehicle compelled to be taken out of the pavement or roadway. Shoulders also act as service lanes for vehicles that have broken down. Refer Fig. 4.4, which gives cross section details of roads in embankment and cutting. The width of shoulder should be adequate to accommodate stationary vehicle fairly away from the edge of adjacent lane. It is desirable to have a minimum shoulder width of 4.6 m so that a truck stationed at the side of the shoulder would have a clearance of 1.85 m from the pavement edge. The minimum shoulder width recommended by the IRC is 2.5 m.

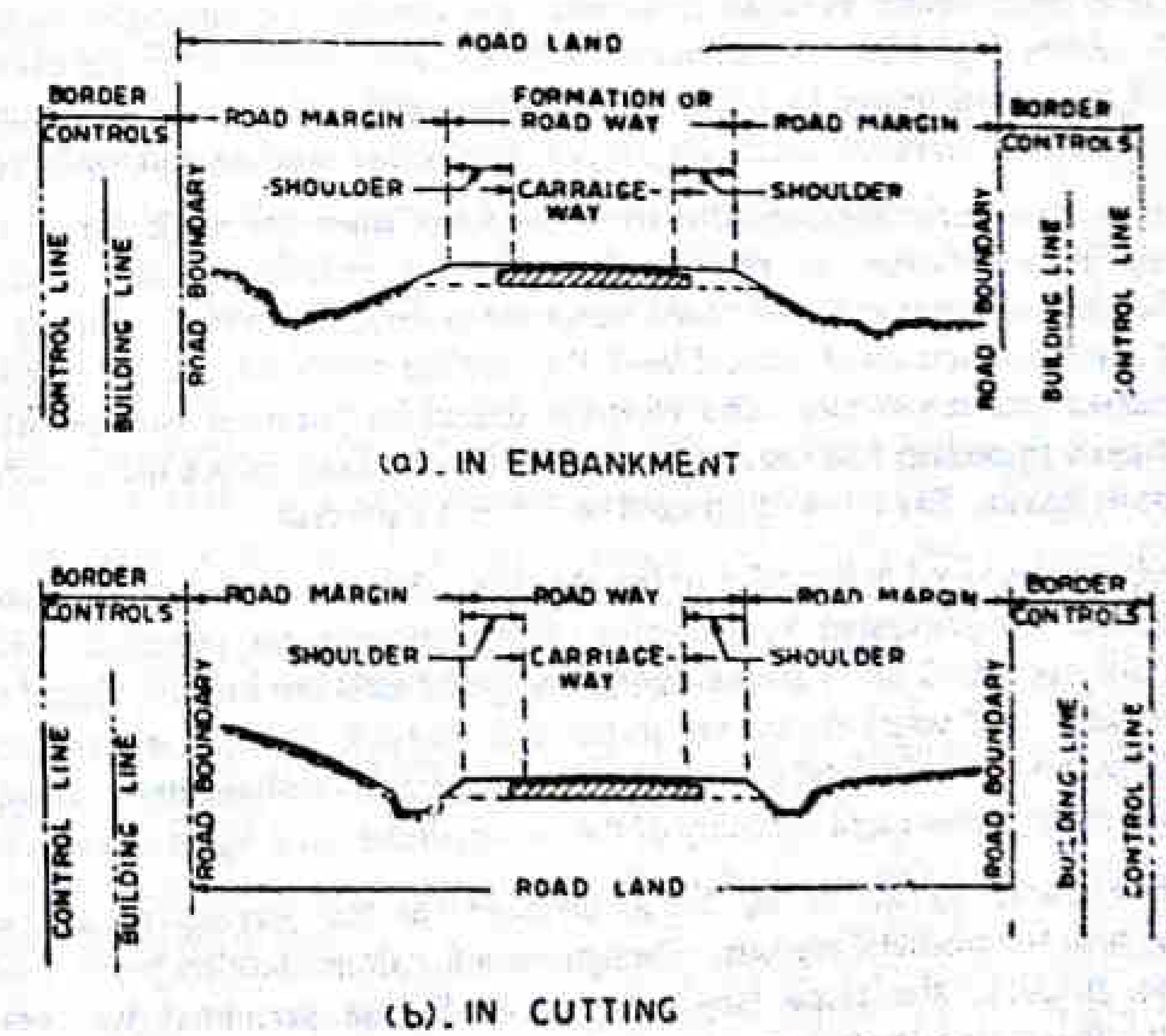


Fig. 4.4 Cross Section Details

The shoulders should have sufficient load bearing capacity to support loaded truck even in wet weather. The surface of the shoulder should be rougher than the traffic lanes so that vehicles are discouraged to use the shoulder as a regular traffic lane. The colour of the shoulder should preferably be different from that of the pavement so as to be distinct.

Parking lanes are provided on urban roads to allow kerb parking. As far as possible only *parallel parking* should be allowed as it is safer for moving vehicles. Also the clearance available between the parked vehicles and the edge of adjacent lane is more in the case of parallel parking than in *angle parking*. The parking lane should also have sufficient width; 3.0 m width is required for parallel parking.

Lay-byes are provided near public conveniences with guide maps to enable drivers to stop clear off the carriageway. Lay-byes should normally be of 3.0 width and atleast 30 m length with 15 m end tapers on both sides.

Bus bays may be provided by recessing the kerb to avoid conflict with moving traffic. Bus bays should be located atleast 75 m away from the intersections.

Frontage roads are provided to give access to properties along an important highway with controlled access to express way or free way. The frontage roads may run parallel to the highway and are isolated by a separator, with approaches to the through facility only at selected points, preferably with grade separations.

Drive ways connect the highway with commercial establishment like fuel-stations, service-stations etc. Drive ways should be properly designed and located, fairly away from an intersection. The radius of the drive way curve should be kept as large as possible, but the width of the drive way should be minimised to reduce the length of cross walks.

Cycle tracks are provided in urban areas when the volume of cycle traffic on the road is very high. Refer Fig. 4.10. A minimum width of 2 m is provided for the cycle track and the width may be increased by 1.0 m for each additional cycle lane. The layout of the cycle tracks should be carefully decided in large highway intersections and traffic rotaries.

Footpath or side walks are provided in urban areas when the vehicular as well as pedestrian traffic are heavy, to provide protection to pedestrians and to decrease accidents. See Fig. 4.3, 4.9 and 4.10. Side walks are generally provided on either side of the road and the minimum width should be 1.5 m and the width may be increased based on the pedestrian traffic volume. The footpath should be provided with a surface as smooth as or even smoother than the adjacent traffic lane so as to induce the pedestrian to keep on to the footpath. The cross fall should be 2.5 to 3.0 percent.

Guard rails are provided at the edge of the shoulder when the road is constructed on a fill so that vehicles are prevented from running off the embankment, especially when the height of the fill exceeds 3 m. Various designs of guard rails are in use. Guard stones (painted with black and white strips) are installed at suitable intervals along the outer edge of the formation at horizontal curves of roads running on embankments along rural areas so as to provide better night visibility of the curves under head lights of vehicles.

Embankment slopes should be as flat as possible for the purpose of safe traffic movement and also for aesthetic reasons. Though from the slope stability point, a steeper slope may be possible, the slope may be kept as flat as permitted by economic considerations. Road side landscaping can improve the aesthetic features of road side, making road travel more pleasant.

4.2.6 Width of Roadway or Formation

Width of formation or roadway is the sum of widths of pavements or carriageway including separators if any; and the shoulders. Formation or roadway width is the top width of the highway embankment or the bottom width of highway cutting excluding the side drains, as shown in Fig. 4.4. The width of roadway standardized by the Indian Roads Congress are given in Table 4.3.

Table 4.3 Width of Roadway of various classes of roads

Sl. No.	Road classification	Roadway width, m at : Plain and Rolling terrain	Roadway width, m at : Mountainous and Steep terrain
1.	National & State Highways		
	(a) Single lane	12.0	6.25
	(b) Two lane	12.0	8.80
2.	Major District Roads		
	(a) Single lane	9.0	4.75
	(b) Two lanes	9.0	-
3.	Other District Roads		
	(a) Single	7.5	4.75
	(b) Two lanes	9.0	-
4.	Village roads-single lane	7.5	4.00

Notes (i) In multilane highways, roadway width should be adequate for the requisite number of traffic lanes besides shoulders and central median.

(ii) The minimum roadway width on single lane bridge is 4.25 m.

4.2.7 Right of Way

Right of way is the area of land acquired for the road, along its alignment. The width of this acquired land is known as *land width* and it depends on the importance of the road and possible future development. A minimum land width has been prescribed for each category of road. A desirable range of land width has also been suggested for each category. While acquiring land for a highway it is desirable to acquire more width of land as the cost of adjoining land invariably increases very much, soon after the new highway is constructed. Also road side developments start taking place making it difficult later on to acquire more land if required for future widening or for other improvements. In some cases the lower width within the suggested range may have to be adopted in view of high cost of land and other existing features. This is particularly true in urban and industrial areas.

The land width is governed by the following factors :

(i) Width of formation depending on the category of highway and width of roadway and road margins.

(ii) Height of embankment or depth of cutting which is governed by the topography and the vertical alignment.

(iii) Side slopes of embankment or cutting which depend on the height of the slope, soil type and several other considerations including aesthetics.

(iv) Drainage systems and their size, which depends on the rainfall, topography, and run off.

(v) Sight distance considerations on horizontal curves, as there is restriction to the visibility on the inner side of the curve due to obstruction such as building structures etc. At sharp curves it is desirable to acquire a wider strip of land in order to avoid obstructions to visibility. Refer Fig. 4.5.

(vi) Reserve land for future widening is to be planned in advance based on anticipated future development and increase in the traffic.

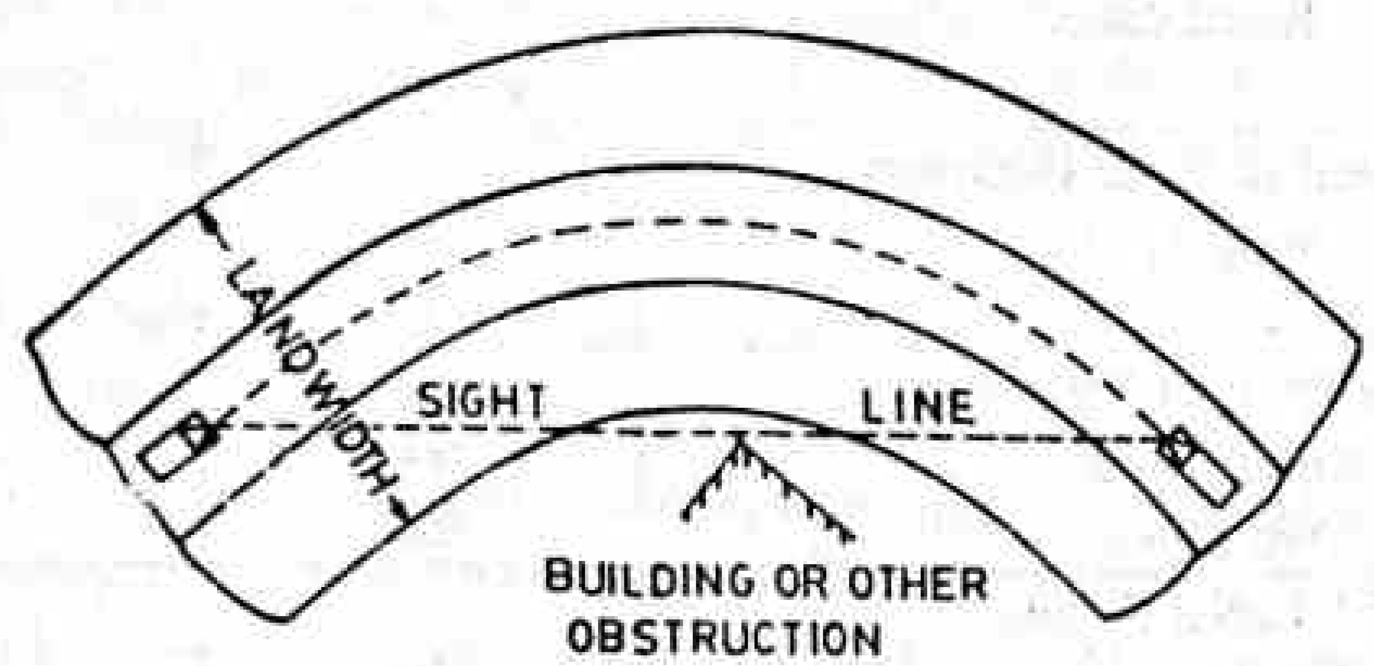


Fig. 4.5 Obstructions to Visibility at Horizontal Curve

The values of normal and range of land width standardized by the IRC for various categories of roads in rural areas and in different terrains are given in Table 4.4 (a).

It is desirable to control the building construction activities on either side of the road boundary, beyond the land width acquired for the road, in order to reserve sufficient space for future improvement of roads. Therefore, it is necessary to disallow the building activities upto "building line" with sufficient setback from the road boundary. In addition, it is desirable so exercise control of the nature of building upto further set back distance upto the "control lines". The overall width requirements between the building lanes and between the control lines on either side of the road, recommended by the IRC for different classification of roads in rural areas at different terrain conditions are given in Table 4.4 (b). It may be seen from Tables 4.4 (a) and 4.4 (b) that the normal land width required for the National and State Highways on open plain terrain is 45 m and the maximum land width required is 60 m, the corresponding width between the building lines is 80 m and that between the control lines is 150 m, thus allowing set back distances of 10 and 45 m beyond the road boundary lines with the maximum recommended road width.

Table 4.4 (a) Recommended land width for different classes of rural roads (metre)

Sl. No.	Road classification	Plain and rolling terrain				Mountainous and steep terrain	
		Open areas		Built-up areas		Open areas	Built-up areas
		Normal	Range	Normal	Range	Normal	Normal
1.	National and State Highways	45	30-60	30	30-60	24	20
2.	Major District Roads	25	25-30	20	15-25	18	15
3.	Other District Roads	15	15-5	15	15-20	15	12
4.	Village Roads	12	12-18	10	10-15	9	9

Table 4.4 (b) Recommended standards for building lines and control lines

Road Classification	Plain and Rolling terrain			Mountainous & steep terrain	
	Open area		Built-up area	Distance between building line and road boundary (set back), m	
	Overall width between building lines, m	Overall width between control lines, m	Distance between building line and road boundary (set-back), m	Open areas	Built-up areas
N.H. & S.H.	80	150	3 to 6	3 to 5	3 to 5
M.D.R.	50	100	3 to 5	3 to 5	3 to 5
O.D.R	25/30*	35	3 to 5	3 to 5	3 to 5
V.R.	25	30	3 to 5	3 to 5	3 to 5

Note : *If the land width is equal to the width between building lines indicated in this column, the building lines should be set back 2.5 m from the road land boundary.

The recommended land widths for different classes of urban roads are, 50 to 60 m for arterial roads (high types of urban roads meant for through traffic, with controlled access), 30 to 40 m for sub-arterial roads, 20 to 30 m for collector streets (urban roads and streets meant for collecting traffic from local streets and feed to the arterial and sub-arterial roads) and 10 to 20 m for local streets.

4.2.8 Typical Cross Sections of Roads

Some of the typical cross sections of rural roads of different categories and urban roads are shown in Fig. 4.6 to 4.10.

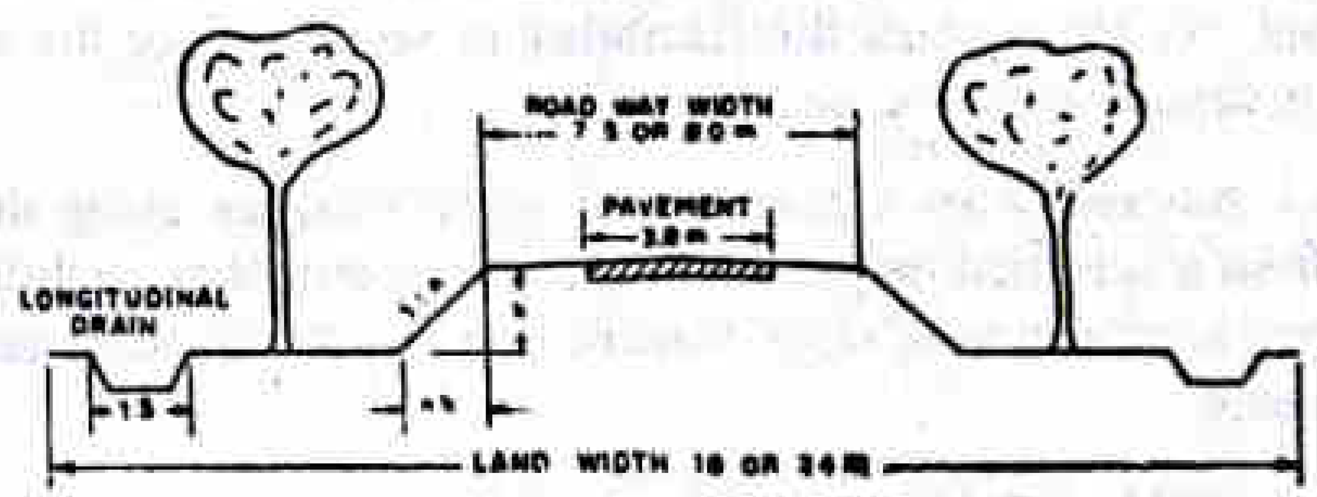


Fig. 4.6 Cross Section of VR or ODR in Embankment in Rural Area

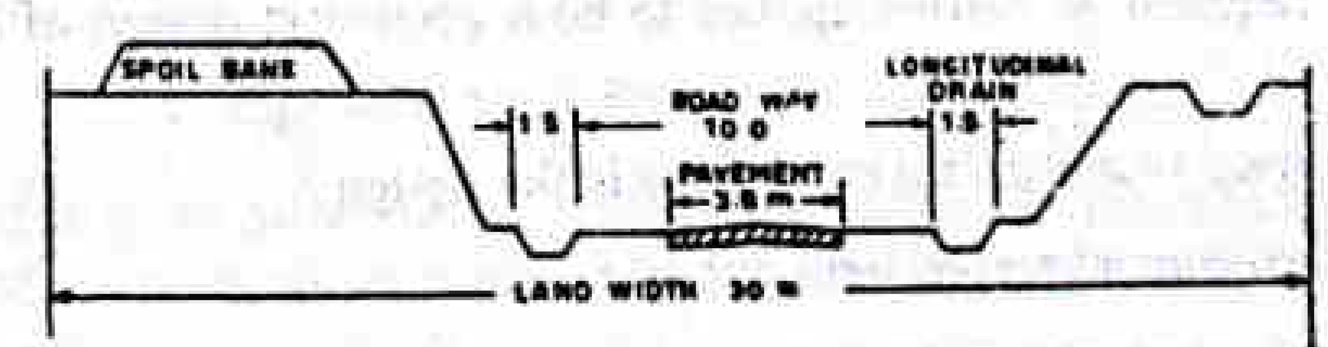


Fig. 4.7 Cross Section of Major District Road in Cutting in Rural Area

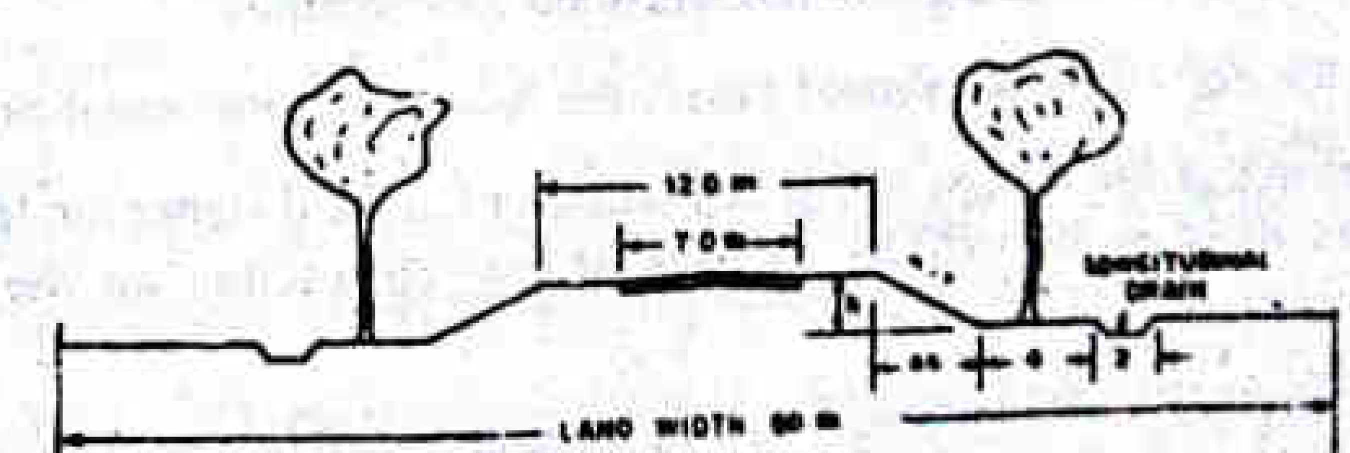


Fig. 4.8 Cross Section of National or SH in Rural Area

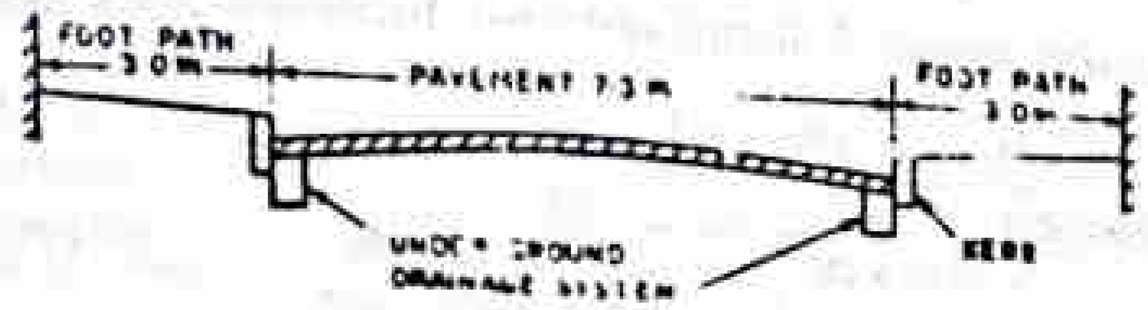


Fig. 4.9 Cross Section of Two-lane City Road in Built-up Area

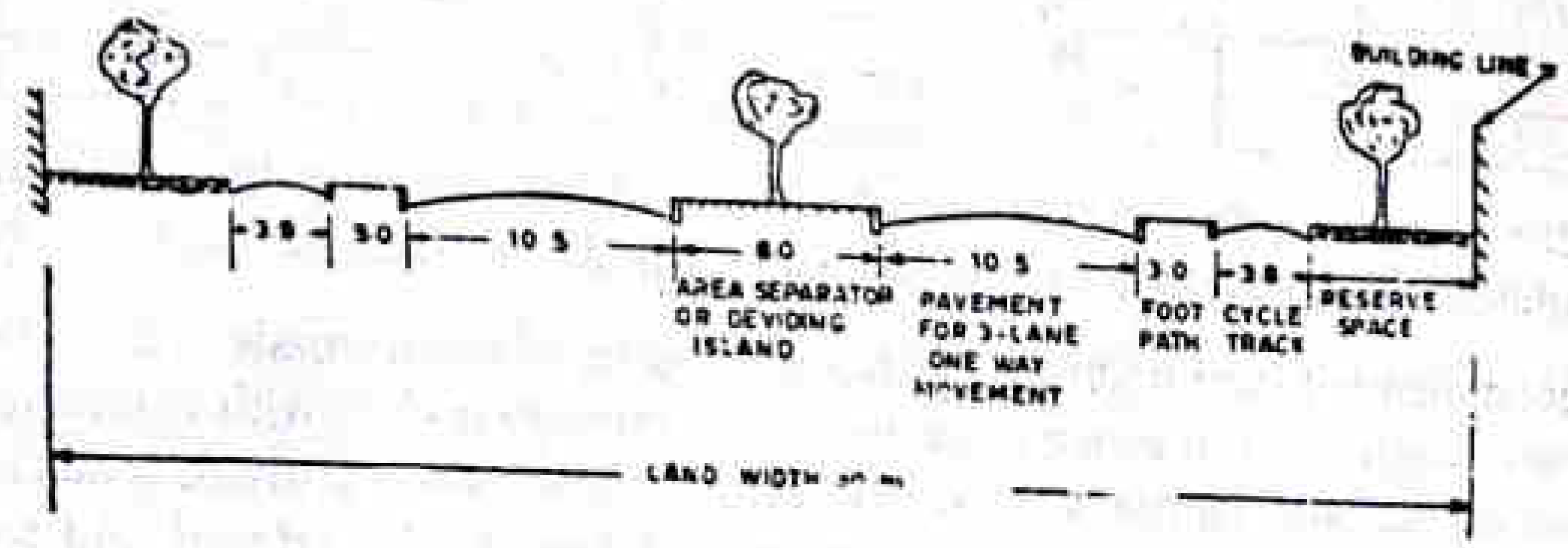


Fig. 4.10 Cross Section of Divided Highway in Urban Area

4.3 SIGHT DISTANCE

4.3.1 Introduction

The safe and efficient operation of vehicle on roads depends, among other factor on the road length at which an obstruction, if any, becomes visible to the driver in the direction of travel. In other words the feasibility to see ahead, or the visibility is very important for safe vehicle operation on a highway.

Sight distance available from a point is the actual distance along the road surface, which a driver from a specified height above the carriageway has visibility of stationary or moving objects. In other words, sight distance is the length of road visible ahead to the driver at any instance.

Restrictions to sight distance may be caused at horizontal curves, by objects obstructing vision at the inner side of the road or at vertical summit curves or at intersections. These are shown in Fig. 4.11.

Sight distance required by drivers applies to both geometric design of highways and for traffic control.

Three sight distance situations are considered in the design :

(i) Stopping or absolute minimum sight distance

(ii) Safe overtaking or passing sight distance, and

(iii) Safe sight distance for entering into uncontrolled intersections

The standards for sight distance should satisfy the following three conditions :

(i) Driver travelling at the design speed has sufficient sight distance or length of road visible ahead to stop the vehicle, in case of any obstruction on the road ahead, without collision.

(ii) Driver traveling at the design speed should be able to safely overtake, at reasonable intervals, the slower vehicles without causing obstruction or hazard to traffic of opposite direction.

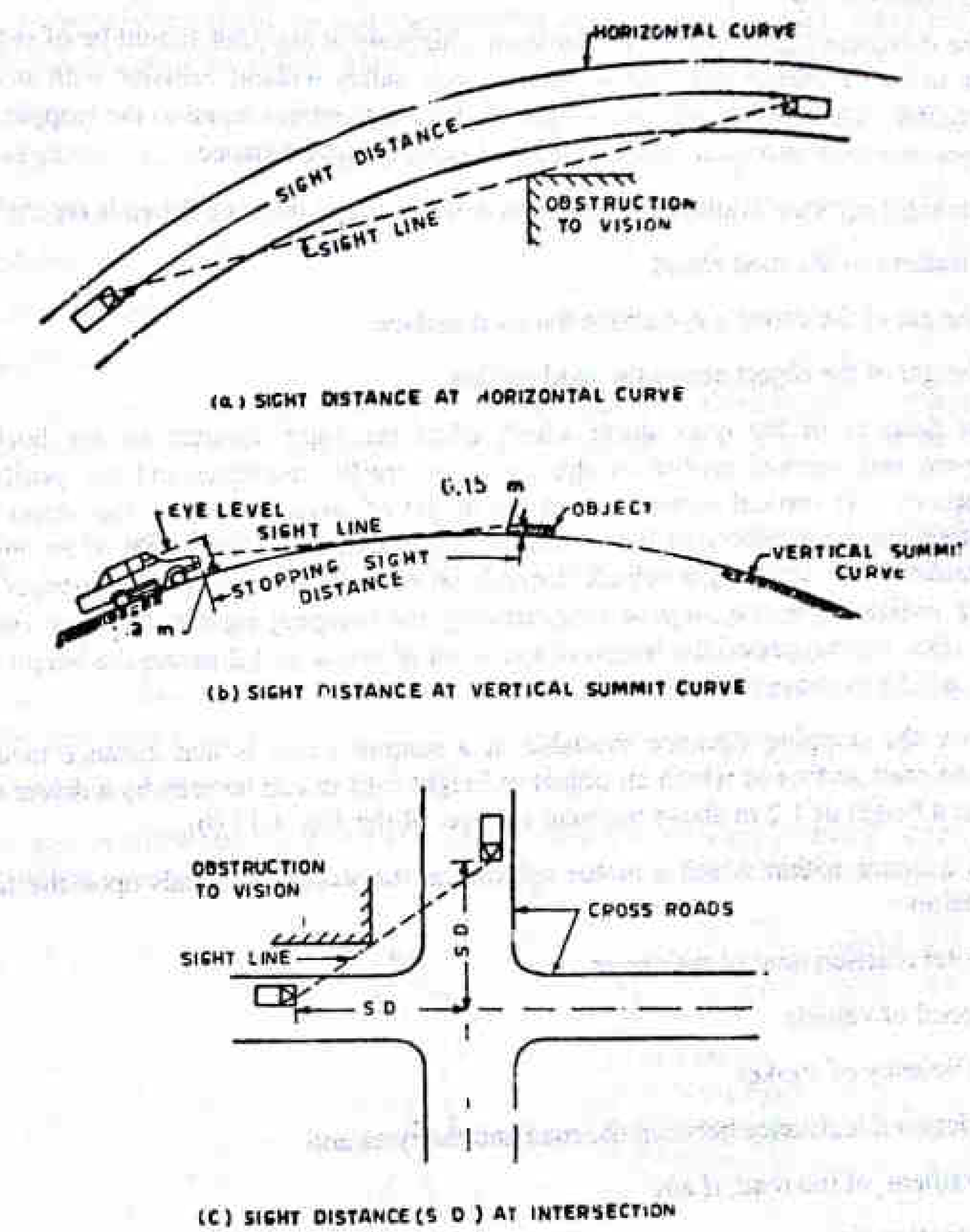


Fig. 4.11 Sight Distance Consideration

(iii) Driver entering an uncontrolled intersection (particularly unsignalised intersection) has sufficient visibility to enable him to take control of his vehicle and to avoid collision with another vehicle.

Apart from the three situations mentioned above, the following sight distances are considered by the IRC in highway design :

(i) *Intermediate sight distance* – This is defined as twice the stopping sight distance. When overtaking sight distance can not be provided, intermediate sight distance is provided to give limited overtaking opportunities to fast vehicles.

(ii) *Head light sight distance* – This is the distance visible to a driver during night driving under the illumination of the vehicle head lights. This sight distance is critical at up-gradients and at the ascending stretch of the valley curves.

4.3.2 Stopping Sight Distance (SSD)

The minimum sight distance available on a highway at any spot should be of sufficient length to stop a vehicle traveling at design speed, safely without collision with any other obstruction. The absolute minimum sight distance is therefore equal to the stopping sight distance, which is also some times called *non-passing* sight distance.

The sight distance available on a road to a driver at any instance depends on

(i) features of the road ahead,

(ii) height of the driver's eye above the road surface.

(iii) height of the object above the road surface.

The features of the road ahead which affect the sight distance are the horizontal alignment and vertical profile of the road, the traffic condition and the position of obstructions. At vertical summit curves the height of driver's eye and the object above road level are more important factors affecting the visibility. The height of an object to be considered for stopping a vehicle depends on what might be a source of danger to the moving vehicle. For the purpose of measuring the stopping sight distance or visibility ahead. IRC has suggested the height of eye level of driver as 1.2 m and the height of the object as 0.15 m above the road surface.

Hence the stopping distance available at a summit curve is that distance measured along the road surface at which an object of height 0.15 m can be seen by a driver where eye is at a height of 1.2 m above the road surface. Refer Fig. 4.11 (b).

The distance within which a motor vehicle can be stopped depends upon the factors listed below :

(a) Total reaction time of the driver

(b) Speed of vehicle

(c) Efficiency of brakes

(d) Frictional resistance between the road and the tyres and

(e) Gradient of the road, if any

Total reaction time

Reaction time of the driver is the time taken from the instant the object is visible to the driver to the instant the brakes are effectively applied. The amount of time gap depends on several factors. During this time the vehicle travels a certain distance at the original speed or the design speed. Thus the stopping distance increases with increase in reaction time of the driver. The total reaction time may be split up into two parts.

(i) perception time

(ii) brake reaction time

The *perception time* is the time required for a driver to realise that brakes must be applied It is the time from the instant the object comes on the line of sight of the driver to the instant he realises that the vehicle needs to be stopped. The perception time varies from driver to driver and also depends on several other factors such as speed of the vehicle, distance of object and other environmental conditions.

The *brake reaction time* also depends on several factors including the skill of the driver, the type of the problems and various other environmental factors. Often the total brake reaction time of the driver is taken together.

PIEV Theory : According to this theory the total reaction time of the driver is split into four parts, viz., time taken by the driver for :

(i) Perception

(ii) Intellection

(iii) Emotion, and

(iv) Volition

Perception time is the time required for the sensations received by the eyes or ears to be transmitted to the brain through the nervous system and spinal chord. In other words, it is the time required to perceive an object or situation.

Intellection time is the time required for understanding the situation. It is also the time required for comparing the different thoughts, regrouping and registering new sensations.

Emotion time is the time elapsed during emotional sensations and disturbance such as fear, anger or any other emotional feelings such as superstition etc. with reference to the situation. Therefore the emotion time of a driver is likely to vary considerably depending upon the problems involved.

Volition time is the time taken for the final action.

It is also possible that the driver may apply brakes or take any avoiding action by the reflex action, even without thinking. The PIEV process has been illustrated in Fig. 4.12.

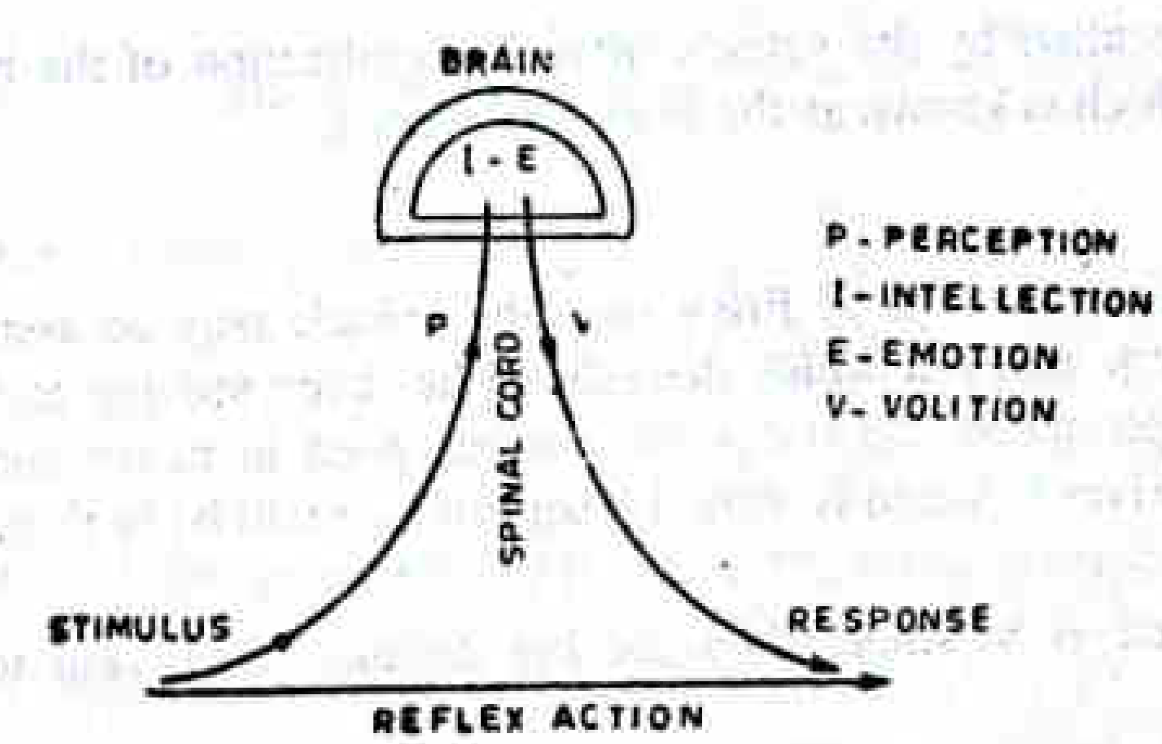


Fig. 4.12 Reaction Time and PIEV Process

The PIEV time of a driver depends on several factors such as physical and psychological characteristics of the driver, type of the problem involved, environmental condition and temporary factors (e.g., motive of the trip, travel speed, fatigue consumption of alcohol, etc.). The total reaction time of an average driver may vary from 0.5 second for simple situations to as much as 3 to 4 seconds or even more in complex problems.

Speed of vehicle

The stopping distance depends very much on the speed of the vehicle. First, during the total reaction time of the driver the distance moved by the vehicle will depend on the speed. Second, the braking distance or the distance moved by the vehicle after applying the brakes, before coming to a stop depends also on the initial speed of the vehicle. Hence it is evident that higher the speed, higher will be the stopping distance.

Efficiency of brakes

The braking efficiency is said to be 100 percent if the wheels are fully locked preventing them from rotating on application of the brakes. This will result in 100 percent skidding which is normally undesirable, except in utmost emergency. Also skidding is considered to be dangerous, as it is not possible to control a skidding vehicle. Hence to avoid skid, the braking forces should not exceed the frictional force between the wheels and tyres.

Frictional resistance between road and tyres

The frictional resistance developed between road and tyres or the *skid resistance* depends on the type and condition of the road surface and the tyres as discussed in article 4.11. The braking distance increases with decrease in skid resistance. IRC has specified a design friction coefficient of 0.35 to 0.4 depending upon the speed to be used for finding the braking distance in the calculation of stopping sight distance. This value, apart from having sufficient safety factor, permits a rate of retardation which is fairly comfortable for passengers.

Analysis of stopping distance

The stopping distance of a vehicle is the sum of :

(i) the distance travelled by the vehicle during the total reaction time know as *lag distance* and

(ii) the distance travelled by the vehicle after the application of the brakes, to a dead stop position which is known as the *braking distance.*

Lag distance

During the total reaction time or PIEV time the vehicle may be assumed to proceed forward with a uniform speed at which the vehicle has been moving and this speed may be taken as the design speed. If 'v' is the design speed in m/sec and 't' is the total reaction time of the driver in seconds, then the lag distance will be 'v.t' metres.

If the design speed is V kmph, then the lag distance works out to $V \times \frac{1000}{60 \times 60}$

t = 0.278 V.t meters.

The total reaction time of driver depends on a variety of factors and a value of 2.5 secs. Is considered reasonable for most situations. The IRC has also recommended the value of reaction time t = 2.5 secs. for the calculation of stopping distance.

Braking distance

The coefficient of friction 'f' depends on several factors such as the type and condition of the pavement surface and tyres. Also the value of f decreases with increase in speed. IRC recommends the following f-values for design :

Speed, kmph	20 to 30	40	50	60	65	80	100
Longitudinal coefficient of friction, f	0.40	0.38	0.37	0.36	0.36	0.35	0.35

Assuming a level road, the braking distance may be obtained by equating the work done in stopping the vehicle and the kinetic energy.

If F is the maximum frictional force developed and the braking distance is l, then work done against friction force in stopping the vehicle is $F \times l = f\,W\,l$, where W is the total weight of the vehicle.

The kinetic energy at the design speed of v m/sec will be

$$\frac{1}{2} m v^2 = \frac{W v^2}{2g}$$

Hence

$$f W l = \frac{W v^2}{2g}$$

or

$$l = \frac{v^2}{2gf}$$

Here l = braking distance, m

v = speed of vehicle, m/sec.

f = design coefficient of friction

= 0.4 to 0.35 depending on speed, from 30 to 80 kmph

g = acceleration due to gravity = 9.8 m/sec^2.

Stopping distance = lag distance + braking distance

i.e.,

$$SD, m = vt + \frac{v^2}{2gf} \quad (4.1)$$

If speed is V kmph, stopping distance

$$SD, m = \left[0.278\, V_t + \frac{V^2}{254 f}\right] \quad (4.2)$$

Equation 4.1 and 4.2 are the general equations for stopping distance at level.

Stopping distance at slopes

When there is an ascending gradient of say, + n% the component of gravity adds to the braking action and hence the braking distance is decreased. The component of gravity acting parallel to the surface which adds to the braking force is equal to W sin α = W tan α = Wn/100.

Equating kinetic energy and work done,

$$\left(fW + \frac{Wn}{100}\right) l = \frac{1}{2}\frac{W v^2}{g}$$

$$l = \frac{v^2}{2g\left(f + \frac{n}{100}\right)}$$

Similarly, in descending gradient of – n% the braking distance increases, as the component of gravity now opposes the braking force. Hence the equation is given by :

$$\left(fW + \frac{Wn}{100}\right) l = \frac{W v^2}{2g}$$

$$l = \frac{v^2}{2g\left(f - \frac{n}{100}\right)}$$

Hence the general equation 4.1 for stopping distance may now be modified for n% gradient and may be written as ·

$$\text{SD, m} = \left[vt + \frac{v^2}{2g(f \pm 0.01n)}\right] \quad (4.3)$$

When the ground is level, n = 0 Eq. 4.3 reduces to Eq. 4.1.

Equation 4.3 may be re-written as follows where the speed is V kmph and the gradient is n percent :

$$\text{SD, m} = 0.278\,V.t + \frac{V^2}{254(f \pm 0.01)n} \quad (4.4)$$

As the Stopping Sight Distance SSD required on descending gradient is higher, it is necessary to determine the critical value of the SSD for the descending gradient on the roads with gradients and two way traffic flow.

The minimum stopping sight distance hence should be equal to the stopping distance in one-way traffic lanes and also in two-way traffic roads when there are two or more traffic lanes. On roads with restricted width or on single lane roads when two-way movement of traffic is permitted, the minimum stopping sight distance should be equal to *TWICE* the stopping distance to enable both vehicle coming from opposite directions to stop. The SSD should invariably be provided throughout the length of all roads and hence this is also known as *absolute minimum sight distance*. When the stopping sight distance for the design speed is not available on any section of a road, the speed should be restricted by a warning sign and a suitable speed-limit regulation sign. However this should be considered only as a temporary measure and wherever possible, the stretch of the road should be re-aligned or the obstruction to visibility removed so as to provide atleast stopping sight distance for the design speed.

The safe stopping distance values calculated in the similar manner for various design speeds and recommended by IRC are given in Table 4.5.

Table 4.5 Stopping sight distance values for different speeds

Design speed, kmph	20	25	30	40	50	60	65	80	100
Safe stopping sight distance for design, m	20	25	30	45	60	80	90	120	180

Example 4.2

Calculate the safe stopping sight distance for design speed of 50 kmph for (a) two-way traffic on a two lane road (b) two way traffic on a single plane road.

Assume coefficient of friction as 0.37 and reaction time of driver as 2.5 seconds

Solution

Stopping distance (Eq. 4.4) = lag distance + braking distance

$$= vt + \frac{v^2}{2gf}$$

$$V = 50 \text{ kmph or } v = \frac{50}{3.6} = 13.9 \text{ m/sec}$$

$$t = 2.5,\ g = 9.8,\ f = 0.37$$

$$\text{Stopping distance} = 13.9 \times 2.5 + \frac{13.9^2}{2 \times 9.8 \times 0.37}$$

$$= 34.8 + 26.6 = 61.4 \text{ m}$$

Alternatively, the stopping distance may also be calculated from Eq. 4.2 as follows :

$$SD = 0.278\,V_t + V^2/254\,f$$

$$= 0.278 \times 50 \times 2.5 + \frac{50^2}{254 \times 0.37} = 61.4 \text{ m}$$

(a) Stopping sight distance when there are two lanes = stopping distance = 61.4 m

(b) Stopping sight distance for two-way traffic with single lane = 2 [stopping distance] = 2 × 61.4 122.8.

Example 4.3

Calculate the minimum sight distance required to avoid a head-on collision of two cars approaching from the opposite directions at 90 and 60 kmph. Assume a reaction time of 2.5 seconds, coefficient of friction of 0.7 and a brake efficiency of 50 percent, in either case.

Solution

Stopping distance for one of the cars (Eq. 4.1).

$$\text{SD metres} = vt + \frac{v^2}{2gf}$$

$$V_1 = 90 \text{ kmph, } v = \frac{90}{3.6} = 25 \text{ m/sec}$$

$$V_2 = 60 \text{ kmph, } v = \frac{60}{3.6} = 16.67 \text{ m/sec}$$

As the brake efficiency is 50%, the wheels will skid through 50% of the braking distance and rotate through the remaining distance. Therefore, the value of coefficient of friction developed f may be taken as 50% of the coefficient of friction, i.e., f = 0.5 × 0.7 = 0.35.

The stopping distance for the first car SD_1

$$= 25 \times 2.5 + \frac{25^2}{2 \times 9.8 \times 0.35} = 153.6 \text{ m}$$

For second car, $$SD_2 = 16.67 \times 2.5 + \frac{16.67^2}{2 \times 9.8 \times 0.35} = 82.2 \text{ m}$$

Sight distance to avoid head-on collision of the two approaching cars

$$= SD_1 + SD_2 = 153.6 + 82.2 = 235.8 \text{ m.}$$

Example 4.4

Calculating the stopping sight distance on a highway at a descending gradient of 2% for a design speed of 80 kmph. Assume other data as per IRC recommendations.

Solution

Total reaction time t may be taken as 2.5 seconds and design coefficient of friction as f = 0.35.

$$V = 80 \text{ kmph; } n = -2\% = -0.02, \ G = 9.8 \text{ m/sec}^2$$

$$v = \frac{80}{3.6} = 22.2 \text{ m/sec}$$

SSD on road with gradient is given in Eq. 4.3 and 4.4.

From Eq. 4.3, $$SSD = vt + \frac{v^2}{2g\,(f \pm n\%)} = 2.2 \times 2.5 + \frac{22.2^2}{2 \times 9.8 (0.35 - 0.02)}$$

$$= 55.5 + 76.2 = 131.7 \text{ m say } 132 \text{ m}$$

Alternatively, using Eq. 4.4

$$SSD = 0.278\ V.t + \frac{V^2}{254\,(f \pm 0.01)\,n}$$

$$= 0.278 \times 80 \times 2.5 + \frac{80^2}{254 (0.35 - 0.02)} = 55.6 + 76.4 = 132 \text{ m}$$

Example 4.5

Calculate the values of (i) Head light sight distance and (ii) Intermediate sight distance for a highway with a design speed of 65 kmph. Assume suitably all the data required.

Solution

$$V = 65 \text{ kmph;}$$

Assume $$f = 0.36, \ t = 2.5 \text{ secs.}$$

...light sight distance $= SSD = 0.278\ Vt + \frac{V^2}{254 f}$

$$= 0.278 \times 65 \times 2.5 + \frac{65^2}{254 \times 0.36} = 91.4 \text{ m}$$

(ii) Intermediate sight distance $= 2\ SSD = 2 \times 91.4 = 182.8$ m

4.3.3 Overtaking Sight Distance (OSD)

If all the vehicles travel on a road at the design speed, then theoretically there should be no need for any overtaking. In fact all vehicles do not move at the designed speed and this is particularly true under mixed traffic conditions. In such circumstances, it is necessary for fast moving vehicles to overtake or pass the slow moving vehicles. It may not be possible to provide the facility to overtake slow moving vehicles throughout the length of a road. In such cases facilities for overtaking slow vehicles with adequate safety should be made possible at frequent distance intervals.

The minimum distance open to the vision of the driver of a vehicle intending to overtake slow vehicle ahead with safety against the traffic of opposite direction is known as the *minimum overtaking sight distance* (OSD) or the *safe passing sight distance* available.

The overtaking sight distance or OSD is the distance measured along the center of the road which a driver with his eye level 1.2 m above the road surface can see the top of an object 1.2 m above the road surface. Refer Fig. 4.13.

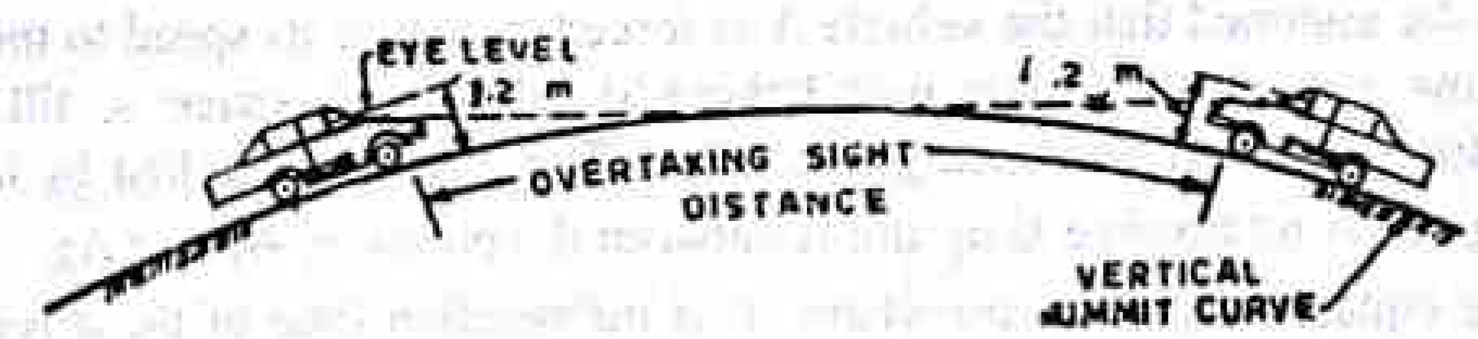


Fig. 4.13 Measurement of Overtaking Sight Distance

Some of the important factors on which the minimum overtaking sight distance required for the safe overtaking manoeuvre depends, are :

(a) speeds of (i) overtaking vehicle (ii) overtaken vehicle and (iii) the vehicle coming from opposite direction, if any.

(b) distance between the overtaking and overtaken vehicles; the minimum spacing depends on the speeds.

(c) skill and reaction time of the driver

(d) rate of acceleration of overtaking vehicle

(e) gradient of the road, if any

Analysis of Overtaking Sight Distance

Figure 4.14 shows the overtaking manoeuvre of vehicle A traveling at design speed, and another slow vehicle B on a two-lane road with two-way traffic. Third vehicle C comes from the opposite direction. The overtaking manoeuvre may be split up into three operations, thus dividing the overtaking sight distance into three parts, d_1, d_2 and d_3.

(i) d_1 is the distance travelled by overtaking vehicle A during the reaction time t sec of the driver from position A_1 to A_2.

(ii) d_2 is the distance travelled by the vehicle A from A_2 to A_3 during the actual overtaking operation, in time T sec.

(iii) d_3 is the distance travelled by on-coming vehicle C from C_1 to C_2 during the overtaking operation of A, i.e. T secs.

Certain assumptions are made in order to calculate the values of d_1, d_2 and d_3.

In Fig. 4.14, A is the overtaking vehicle originally traveling at design speed v m/sec or V kmph; B is the overtaken or slow moving vehicle moving with uniform speed v_b m/sec or V_b kmph; C is a vehicle coming from opposite direction at the design speed v m/sec or V kmph. In a two-lane road the opportunity to overtake depends on the frequency of vehicles from the direction and the overtaking sight distance available at any instant.

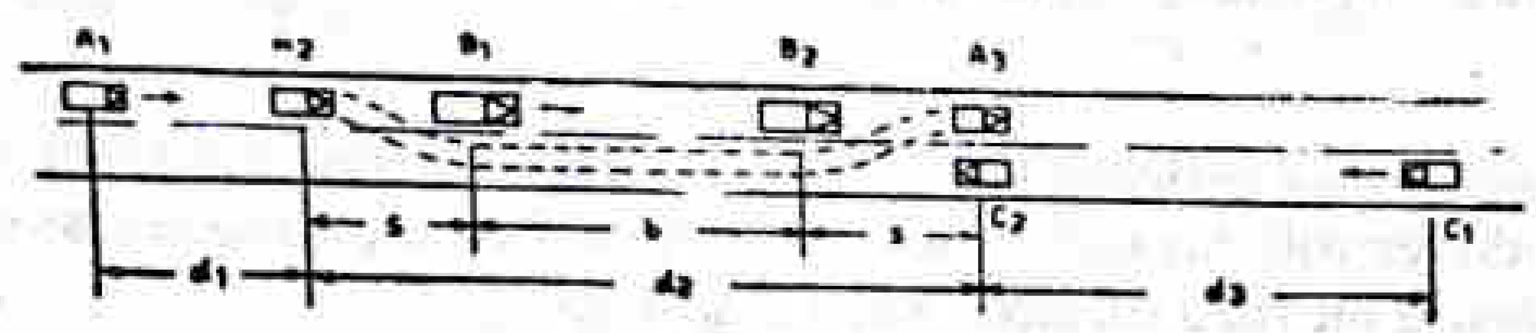


Fig. 4.14 Overtaking Manoeuvre

(i) It may be assumed that the vehicle A is forced to reduce its speed to the speed v_b of the slow vehicle B and moves behind it allowing a space s, till there is an opportunity for safe overtaking operation. The distance travelled by the vehicle A during this reaction time is d_1 and is between the positions A_1 and A_2. This distance will be equal to $v_b \times t$ metre where 't' is the reaction time of the driver in second. This reaction time 't' of the driver may be taken as two seconds as an average value, as the aim of the driver is only to find an opportunity to overtake. Thus,

$$d_1 = v_b\, t = 2\, v_b,\ \text{m}$$

(ii) From position A_2, the vehicle A starts accelerating, shifts to the adjoining lane, overtakes the vehicle B, and shifts back to it original lane ahead of B in position A_3 in time T sec. The straight distance between position A_2 and A_3 is taken as d_2. The minimum distance between position A_2 and B_1 may be taken as the minimum spacing 's' of the two vehicles while moving with the speed v_b m/sec. The minimum spacing between vehicles depends on their speed and is given by empirical formula :

$$s = (0.7\, v_b + 6),\ \text{m}$$

The minimum distance between B_2 and A_3 may also be assumed equal to s as mentioned above. If the time taken by vehicle A for the overtaking operation from position A_2 to A_3 is T second, the distance covered by the slow vehicle B traveling at a speed of v_b m/sec. = b = v_b/T m.

Now the time T depends on speed of overtaken vehicle B and the acceleration of overtaking vehicle A. This time T may be calculated by equating the distance d_2 to (v_b T + ½ a T^2), using the general formula for the distance travelled by an uniformly accelerating body with initial speed v_b m/sec and 'a' is the acceleration in m/sec^2.

$$d_2 = (b + 2s) = \left(v_b\, T + \frac{aT^2}{2}\right)$$

$$b = vb.T, \text{ and therefore } 2s = \frac{aT^2}{2}$$

Therefore, $$T = \sqrt{\frac{4s}{a}} \text{ sec, where } s = (0.7\, v_b + 6)$$

Hence, $$d_2 = (v_b\, T + 2s),\ \text{m}$$

(iii) The distance travelled by vehicle C moving at design speed v m/sec during the overtaking operation of vehicle A i.e. during time T is the distance d_3 between positions C_1 to C_2.

Hence, $$d_3 = v \times T$$

Thus the overtaking sight distance

$$OSD = (d_1 + d_2 + d_3)$$

$$= (v_b\, t + v_b\, T + 2s + vT) \qquad (4.5)$$

In kmph units, equations (4.5) worksout as :

$$OSD = 0.28\, V_b\, t + 0.28\, V_b\, T + 2s + 0.28\, V.T \qquad (4.6)$$

Here V_b = speed of overtaking vehicle, kmph

t = reaction time of driver = 2 secs.

V = speed of overtaking vehicle or design speed, kmph

$$T = \sqrt{\frac{4 \times 3.6s}{A}} = \sqrt{\frac{14.4s}{A}}$$

s = spacing of vehicles = (0.2 V_b + 6)

A = acceleration, kmph/sec.

In case the speed of overtaken vehicle V_b is not given, the same may be assumed as (V – 16) kmph where V is the design speed in kmph or v_b = (v – 4.5) m/sec and v is the design speed in m/sec.

The acceleration of the overtaking vehicle is to be specified. Usually this depends on the make of the vehicle, its condition, load and the speed. As a general guide Table 4.6 may be used for finding the maximum acceleration of vehicles at different speeds. The average rate of acceleration during overtaking manoeuvre may be taken corresponding to the design speed.

Table 4.6 Maximum overtaking acceleration at different speeds

Speed		Maximum overtaking acceleration	
V, kmph	v, m/sec	A, kmph/sec	a, m/sec^2
25	6.93	5.00	1.41
30	8.34	4.80	1.30
40	11.10	4.45	1.24
50	13.86	4.00	1.11
65	18.00	3.28	0.92
80	22.20	2.56	0.72
100	27.80	1.92	0.53

At overtaking sections, the minimum overtaking distance should be ($d_1 + d_2 + d_3$) when two-way traffic exists. On divide highways and on roads with one way traffic regulation, the overtaking distance need be only ($d_1 + d_2$) as no vehicle is expected from the opposite direction. On divided highways with four or more lanes, IRC suggests that it is not necessary to provide the usual OSD; however the sight distance on any highway should be more than the SSD, which is the absolute minimum sight distance.

Effect of grade in overtaking sight distance

Appreciable grades on the road, both the descending as well as ascending, increase the sight distance required for safe overtaking. In down grades though it is easier for the overtaking vehicles to accelerate and pass the overtaken vehicle may also accelerate and cover a greater distance 'b' during the overtaking time.

On up grades, the acceleration of the overtaking vehicle will be less and hence passing will be difficult; but the overtaken vehicle like heavily loaded trucks may also decelerate at steep ascends and compensate to some extent the passing sight distance requirement. Therefore the OSD at both ascending and descending grades are taken as equal to that at level stretch. However, at grades the overtaking sight distance should be greater than the minimum overtaking distance required at level.

The IRC has specified the safe values of overtaking sight distance required for various design speeds between 40 and 100 kmph. These values have been suggested based on the observation that 9 to 14 seconds are required by the overtaking vehicle for the actual overtaking manoeuvre depending on the design speed. This overtaking time may be increased by about two-third to take into account the distance covered by the vehicle from the opposing direction in the case of two-way traffic road, during the overtaking operation. The OSD values thus obtained for various design speeds are rounded off by the IRC and tne recommended values of OSD on two lane highways are given in Table 4.7.

Table 4.7 Overtaking sight distance on two-lane highways for various speeds

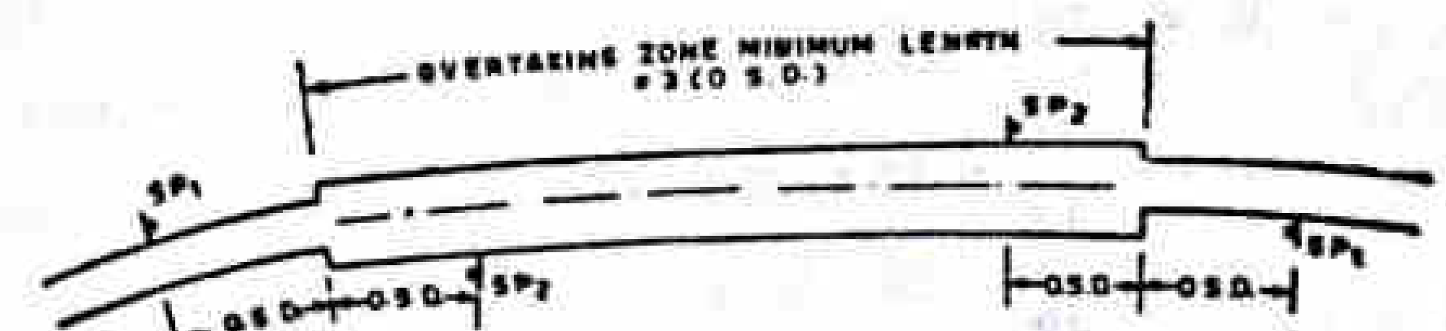
Speed kmph	Time component, seconds			Safe overtaking sight distance (metres)
	For overtaking manoeuvre	For opposing vehicle	Total	
40	9.0	6.0	15	165
50	10.0	7.0	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14.0	9.0	23	640

Overtaking Zones

It is desirable to construct highways in such a way that the length of road visible ahead at every point is sufficient for safe overtaking. This is seldom practicable and there may be stretches where the safe overtaking distance can not be provided. In such zones where overtaking or passing is not safe or is not possible, sign posts should be installed indicating "*No Passing*" or "*Overtaking Prohibited*" before such restricted zones starts. But the overtaking opportunity for vehicles moving at design speed should be given at frequent intervals. These zones which are meant for overtaking are called *overtaking zones*.

The OSD and pavement width should be sufficient for safe overtaking operations. Sign posts should be installed at sufficient distance in advance to indicate the start of the overtaking zones; this distance may be equal to ($d_1 + d_2$) for one-way roads and ($d_1 + d_2 + d_3$) for two-way roads. Similarly the end of the overtaking zones should also be indicated by appropriate sign posts installed ahead at distances specified above. The minimum length of overtaking zone should be three time the safe overtaking distance i.e., 3 ($d_1 + d_2$) for one-way roads and 3 ($d_1 + d_2 + d_3$) for two-way roads. It is desirable that the length of overtaking zones is kept *five* times the overtaking sight distance.

Figure 4.15 shows an overtaking zone with specifications for the positions of the sign posts.



O.S.D = Overtaking sight distance
= ($d_1 + d_2$) for one way traffic
= ($d_1 + d_2 + d_3$) for two way traffic
SP_1 = Sign post "over taking zone ahead"
SP_2 = Sign post "end of overtaking zone"

Fig. 4.15 Overtaking Zones

Criteria for Sight Distance Requirements on Highway

The absolute minimum sight distance required throughout the length of the road is the SSD which should invariably be provided at all places. On horizontal curves the obstruction on the inner side of the curves should be cleared to provide the required set back distance and absolute minimum sight distance. The common obstruction to clear vision on horizontal curves are buildings and other structures, trees, advertisement boards, cut slopes, etc. On vertical summit curves the sight distance requirement may be fulfilled by proper design of the vertical alignment as given in Article 4.5. At uncontrolled intersections sufficient clearances to the sight lines may be given to provide for SSD.

Intermediate Sight Distance

Sufficient overtaking sight distance should be available on most of the road stretches. On horizontal curves the overtaking sight distance requirements can not always be fulfilled especially on sharp curves, if the safe overtaking sight distance requirements are high. In such cases overtaking should be prohibited by regulatory signs. In case of vertical summit curves, it is possible to provide the sight distance requirements by suitably designing the vertical alignment. At stretches of the road where required overtaking sight distance can not be provided as per Table 4.7, as far as possible *Intermediate Sight Distance*, ISD equal to twice SSD may be provided. (Refer Table 4.5). The measurement of the ISD may be made assuming both the height of the eye level of the driver and the object to be 1.2 metres above the road surface.

Example 4.6

The speed of overtaking and overtaken vehicles are 70 and 40 kmph, respectively on a two way traffic road. If the acceleration of overtaking vehicle is 0.99 m/sec^2.

(a) calculate safe overtaking sight distance

(b) mention the minimum length of overtaking zone and

(c) draw a neat-sketch of the overtaking zone and show the positions of the sign posts.

Solution

(a) Overtaking sight distance for two way traffic

$$= d_1 + d_2 = d_3 \quad (4.5)$$

Assume the design speed as the speed of overtaking vehicle A

$$V = 70 \text{ kmph}$$

$$v = \frac{70}{3.6} = 19.4 \text{ m/sec}$$

$$v_b = \frac{40}{3.6} = 11.1 \text{ m/sec}$$

Acceleration, $a = 0.99$ m/sec per sec.

$$D_1 = v_b.\,t \text{ (Adopt } t = 2 \text{ secs)} = 11.1 \times 2 = 22.2 \text{ m}$$

$$d_2 = v_b.T + 2.s$$

$$s = (0.7\, v_b + 6) = (0.7 \times 11.1 + 6) = 13.8 \text{ m}$$

$$T = \sqrt{\frac{4.s}{a}} = \sqrt{\frac{4 \times 13.8}{0.99}} = 7.47 \text{ secs}$$

$$d_2 = 11.1 \times 7.47 + 2 \times 13.8 = 110.5 \text{ m}$$

$$d_3 = v.\,T = 19.4 \times 7.47 = 144.9 \text{ m}$$

$$\text{O.S.D} = d_1 + d_2 + d_3$$

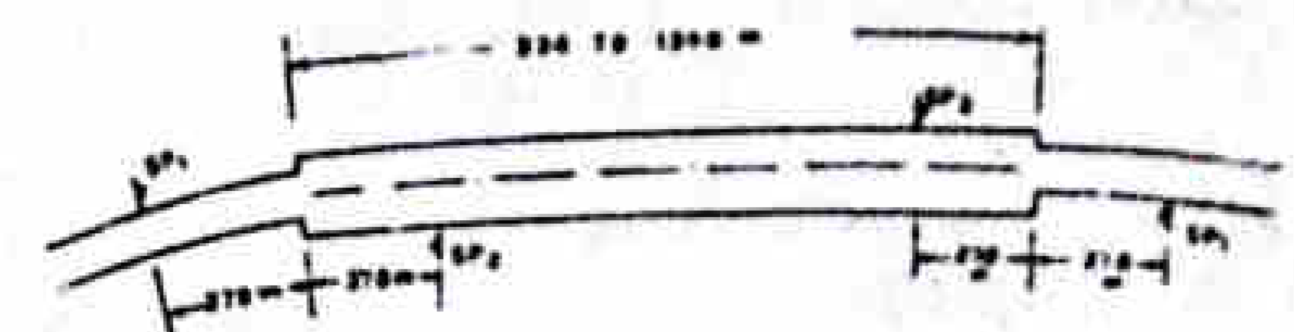
$$= 22.2 + 110.5 + 144.9 = 277.6 \text{ m, say } 278 \text{ m}$$

(b) Minimum length of overtaking zone = 3 (OSD)

$= 3\,(d_1 + d_2 + d_3)$ for two-way traffic $= 3 \times 278 = 834$ metres

Desirable length of overtaking zone = 5 × (OSD) = 5 × 278 = 1390 m

(c) The details of the overtaking zone are shown in Fig. 4.16.



SP_1 – Sign Post "Overtaking Zone Ahead"

SP_2 – Sign Post "End of Overtaking Zone"

Fig. 4.16 Overtaking Zone (Example 4.6)

Example 4.7

Calculate the safe overtaking sight distance for a design speed of 96 kmph. Assume all other data suitably.

Solution

$$\text{O.S.D} = (d_1 + d_2) \text{ for one-way traffic}$$

$$= (d_1 + d_2 + d_3) \text{ for two-way traffic}$$

$$V = 96 \text{ kmph}$$

$$\text{Assume } V_b = V - 16 = 80 \text{ kmph and}$$

A 2.5 kmph/sec. (from Table 4.7); $t = 2$ secs.

$$d_1 = 0.28\, V_b\, t = 0.28 \times 80 \times 2 = 44.8 \text{ m}$$

$$d_2 = 0.28\, V_b\, T + 2.s$$

$$s = (0.2\, V_b + 6) = 0.2 \times 80 + 6 = 22 \text{ m}$$

$$T = \sqrt{\frac{14.4\,s}{A}} = \sqrt{\frac{14.4 \times 22}{2.5}} = 11.3 \text{ secs.}$$

$$d_2 = 0.28 \times 80 \times 11.3 + 2 \times 22 = 297 \text{ m}$$

$$d_3 = 0.28\, V\, T = 0.28 \times 96 \times 11.3 = 303.7 \text{ m}$$

O.S.D. on one-way traffic road $= d_1 + d_2 = 341.8$ m; say 342 m

O.S.D. on two-way traffic road $= d_1 + d_2 + d_3 = 645.5$ m; say 646 m

4.3.4 Sight Distance at Intersections

It is important that on all approaches of intersecting roads, there is a clear view across the corners from a sufficient distance so as to avoid collision of vehicles. This is all the more important at uncontrolled intersections. The sight line is obstructed by structures or other objects at the corners of the intersections. The area of unobstructed sight formed by the lines of vision is called the sight triangle. See Fig. 4.17.

The design of sight distance at intersections may be based on three possible conditions :

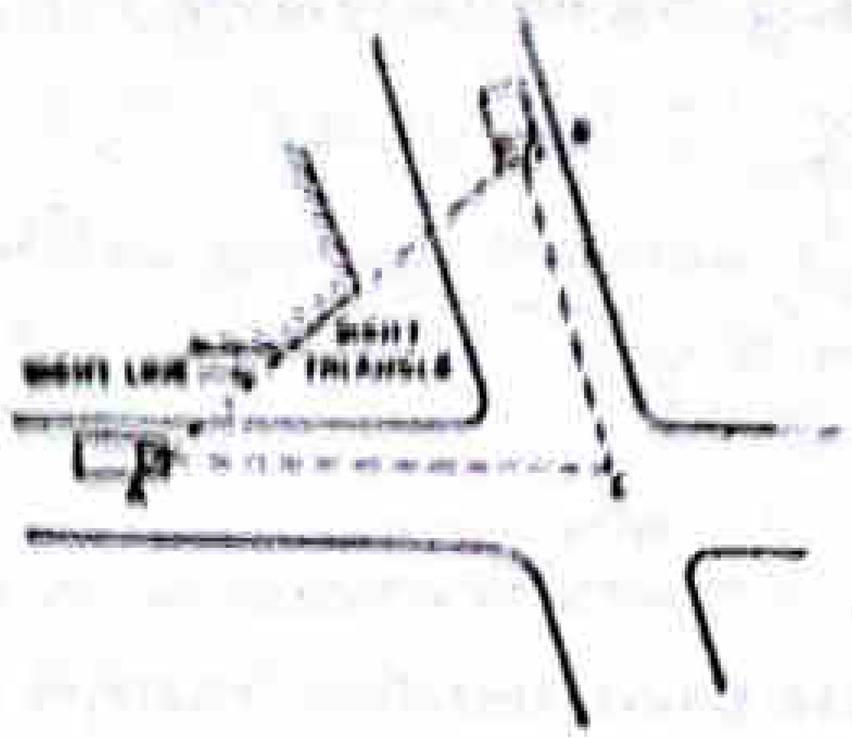


Fig. 4.17 Sight Distance at Intersection

(i) *Enabling the approaching vehicle to change speed* : The sight distance should be sufficient to enable either one or both the approaching vehicles to change speed to avoid collision. The vehicle approaching from the minor road should slow down. The total reaction time required for the driver to decide to change speed may be assumed as two seconds and at least one more second will be needed for making the change in speed. Hence the two sides AC and BC of the sight triangle along the intersection approaches upto the conflict point C should be atleast equal to the distance covered by a vehicle traveling at design speed in two seconds. But this sight distance being too less, should be increased in all possible cases.

(ii) *Enabling approaching vehicle to stop* : In this case, the distances for the approaching vehicle should be sufficient to bring either one or both of the vehicle to a stop before reaching a point of collision. Hence the two sides AC and BC of the sight triangle should each be equal to the safe stopping distance. In almost all uncontrolled intersections one of the two cross roads is a preference highway or a through road or a major road. Thus it is the responsibility of the drivers on the minor road who would cross or enter this main road, to stop or change speed to avoid collision. The traffic of the minor road is generally controlled by an appropriate traffic sign. In such a case the sight distance for a minor road should be atleast equal to the SSD for the design speed of that road. The sight distance requirement of stopping is higher than that of condition (i) above and hence is safe as vehicles can stop if necessary.

(iii) *Enabling stopped vehicle to cross a main road* : This case is applicable when the vehicles entering the intersection from the minor road are controlled by stop sign and so these vehicles have to stop and then proceed to cross the main road. In such a situation, the sight distance available from the stopped position of the minor road should be sufficient to enable the stopped vehicle to start, accelerate and cross the main road, before another vehicle travelling at its design speed on the main road reaches the intersection. The time T required for the stopped vehicle to cross the main road would depend upon (a) reaction time of the driver (b) width of the main road (c) acceleration, and (d) length of vehicle. Thus the minimum sight distance to fulfil this condition is the distance travelled by a vehicle on the main road at design speed during this time 'T'.

From safety considerations, the sight distance at *uncontrolled intersections* should *therefore* fulfil all the above three conditions. The higher of the three values may be *taken* at unsignalised intersections at grade, except at rotaries. The IRC recommends that

at uncontrolled intersections, sufficient visibility should be provided such that the sight distance of each road is atleast equal to the SSD corresponding to the design speed of the road. If the sight triangle available is less than the desirable minimum size due to unavoidable reasons, the vehicles approaching the intersection may be warned or controlled by suitable signs.

At rotaries the sight distance should be at least equal to the safe stopping distance for the design speed of the rotary. At signalized intersections, the above three requirements are not applicable.

At *priority intersections* where a minor road crosses a major road, the traffic on the minor road may be controlled by stop or give-way sign to give priority to the traffic on the major road. The visibility distance available along the minor road should be sufficient to enable the drivers stop their vehicles. The visibility distance along the major road depends upon the time required for the stopped vehicles approaching from the minor road to evaluate the gaps between the vehicles on the major road, to accelerate and to cross the major road safety. IRC recommends that a minimum visibility distance of 15 m along the minor road and a distance of 220, 180, 145 and 110m along the major and corresponding to the design speeds of 100, 80, 65 and 50 kmph respectively may be provided.

4.4 DESIGN OF HORIZONTAL ALIGNMENT

4.4.1 General

Often changes in the direction are necessitated in highway alignment due to obligatory points as discussed in Chapter 3. Various design factors to be considered in the horizontal alignment are design speed, radius of circular curves, type and length of transition curves, superelevation and widening of pavement on curves.

The alignment should enable consistent, safe and smooth movement of vehicles operating at design speeds. It is hence necessary to avoid those sharp curves and reverse curves which could not be conveniently negotiated by the vehicles at design speed. Improper design of horizontal alignment of roads would necessitate speed changes resulting in increased vehicle operation cost and higher accident rate.

4.4.2 Design Speed

The overall design of geometrics of any highway is a function of the design speed.

The design speed is the main factor on which geometric design elements depends. The sight distances, radius of horizontal curve, superelevation, extra widening of pavement, length of horizontal transition curve and the length of summit and valley curve are all dependent on design speed.

The design speed of roads depends upon (i) class of the road and (ii) terrain. The speed standards of a particular class of road thus depends on the classification of the terrain through which it passes. The terrains have been classified as plain, rolling, mountainous and steep, depending on the cross slope of the country as given below :

Terrain classification	Cross slope of the country, percent
Plain	0-10
Rolling	10-25
Mountainous	25-60
Steep	greater than 60

The design speed (ruling and minimum) standardized by the IRC for different classes of roads on different terrains in rural areas are given in Table 4.8. The ruling design speeds are the guiding criteria for the geometric design. However, minimum design speeds may be accepted where site conditions or economic considerations warrant. The ruling design speeds suggested for the National and State Highways of our country passing through plain terrain is 100 kmph and through rolling terrain is 80 kmph.

Table 4.8 Design Speeds on Rural Highways

Road classification	Design speed in kmph for various terrains							
	Plain		Rolling		Mountainous		Steep	
	Ruling	Min.	Ruling	Min.	Ruling	Min.	Ruling	Min.
National & State Highways	100	80	80	65	50	40	40	30
Major District Roads	80	65	65	50	40	30	30	20
Other District Roads	65	50	50	40	30	25	25	20
Village Roads	50	40	40	35	25	20	25	20

Speed restrictions have been imposed for heavy vehicles (other than passenger cars) like buses, trucks and vehicles pulling trailer units under *Motor Vehicles Act*. Also speed limits are specified for different categories of vehicles by regulatory signs on urban roads and on some stretches of rural highway when warranted due to safety considerations.

The recommended design speeds for different classes of urban roads are :

(i) for arterial roads 80 kmph,

(ii) sub-arterial roads 60 kmph,

(iii) collector streets 50 kmph and

(iv) local streets 30 kmph

4.4.3 Horizontal Curves

A horizontal highway curve is a curve in plan to provide change in direction to the central line of a road. When a vehicle traverses a horizontal curve, the centrifugal force acts horizontally outwards through the centre of gravity of the vehicle.

The centrifugal force developed depends on the radius of the horizontal curves and the speed of the vehicle negotiating the curve. This centrifugal force is counteracted by the transverse frictional resistance developed between the tyres and the pavement which enables the vehicle to change the direction along the curve and to maintain the stability of the vehicle. Centrifugal force P is given by the equation :

$$P = \frac{W v^2}{gR}$$

Here
- P = centrifugal force, kg
- W = weight of the vehicle, kg
- R = radius of the circular curve, m
- v = speed of vehicle, m/sec
- ... gravity = 9.8 m/sec^2

The ratio of the centrifugal force to the weight of the vehicle, P/W is known as the *centrifugal ratio* or the *impact factor*. The centrifugal ratio is thus equal to v^2/gR.

The centrifugal force acting on a vehicle negotiating a horizontal curve has two effects :

(i) *Tendency to overturn the vehicle outwards about the outer wheels and*

(ii) *Tendency to skid the vehicle laterally, outwards.*

The analysis of stability of those two conditions against overturning and transverse skidding of the vehicles negotiating horizontal curves without superelevation are given below :

(i) Overturning effect

The centrifugal force that tends the vehicle to overturn about the outer wheels B on horizontal curve without superelevation is illustrated in Fig. 4.18. The overturning moment due to centrifugal force P is P × h; this is resisted by the restoring moment due to weight of the vehicle W and is equal to W.b/2, where h is the height of the center of gravity of the vehicle above the road surface and b is the width of the wheel base or the wheel track of the vehicle.

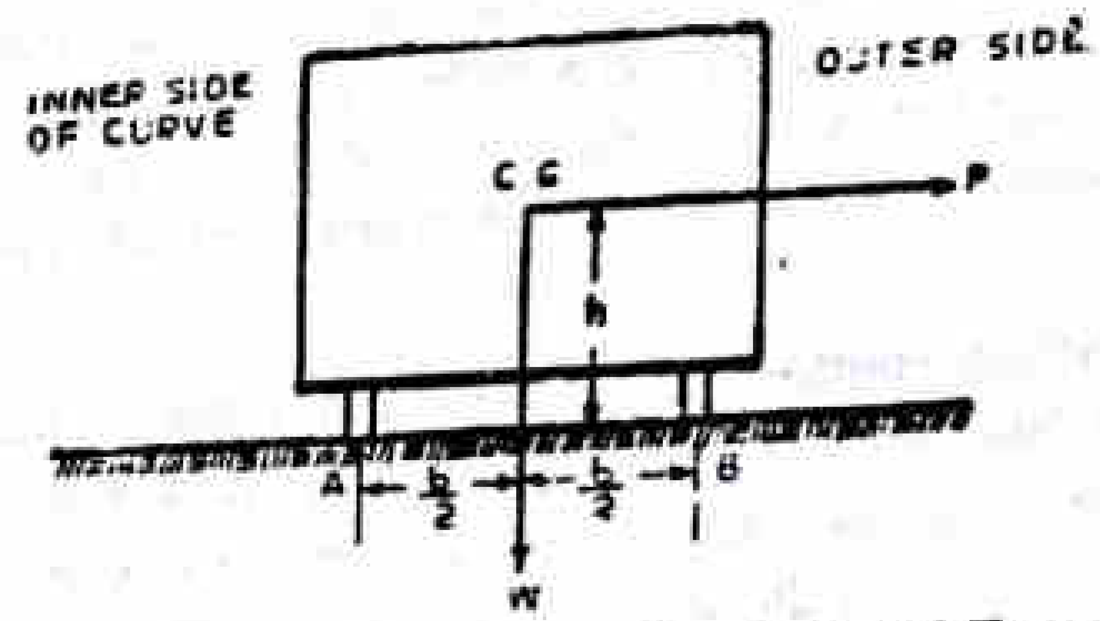


Fig. 4.18 Overturning due to Centrifugal Force

The equilibrium condition for overturning will occur when Ph = Wb/2, or when P/W = b/2h. This means that there is danger of overturning when the centrifugal ratio P/W or v^2/gR attains a values of b/2h.

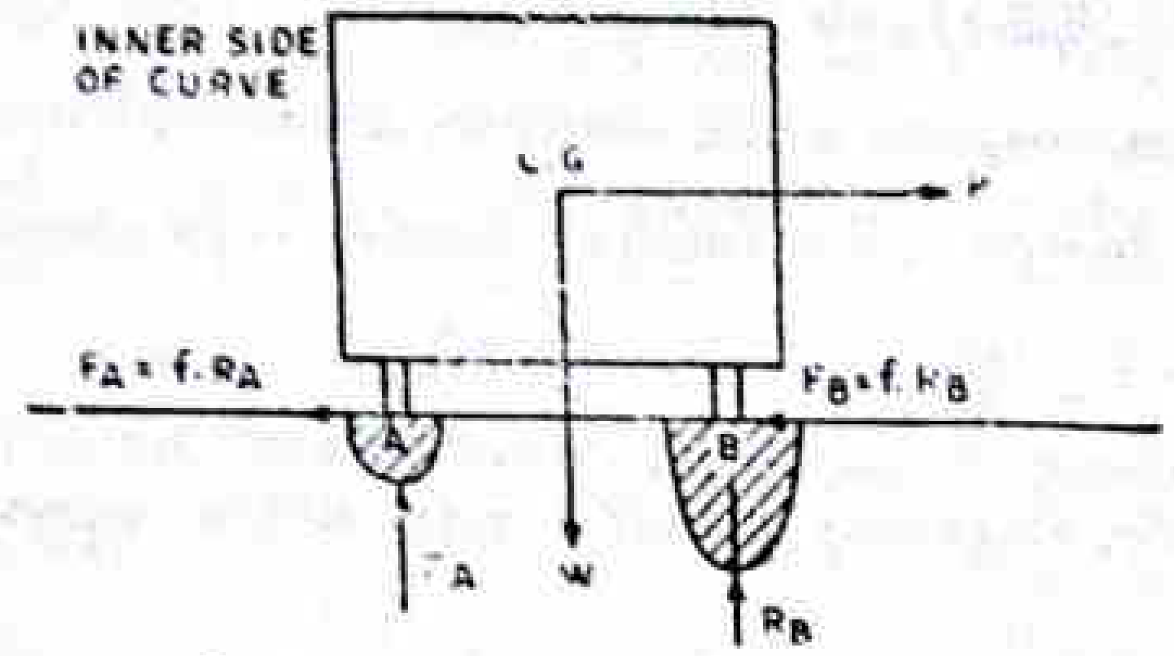
(ii) Transverse skidding effect

The centrifugal force developed has also the tendency to push the vehicle outwards in the transverse direction. If the centrifugal force P developed exceeds the maximum possible transverse skid resistance due to the friction, the vehicle will start skidding in the transverse direction. Refer Fig. 4.19. The equilibrium condition for the transverse skid resistance developed is given by :

$$P = F_A + F_B = f(R_A + R_B) = fW$$

In the above relation, f is the coefficient of friction between the tyre and the pavement surface in the transverse direction, R_A and R_B are normal reactions at the wheels A and B such that $(R_A + R_B)$ is equal to the weight W of the vehicle, as no superelevation has been provided in this case.

Since P = f W, the centrifugal ratio P/W is equal to 'f'. In other words when the centrifugal ratio attains a value equal to the coefficient of lateral friction there is a danger of lateral skidding.



SHADED AREAS SHOW THE PRESSURE UNDER THE INNER AND OUTER WHEELS A AND B

Fig. 4.19 Skidding Effect due to Centrifugal Force

Thus to avoid overturning and lateral skidding on a horizontal curve, the centrifugal ratio should always be less than b/2h and also 'f'.

The vehicle negotiating a horizontal curve with no superelevation has to fully depends on the coefficient of friction 'f' to resist the lateral skidding. The centrifugal force may be enough to cause overturning or lateral skidding of the vehicle if either the speed of the vehicle is high or the radius of the curve is less. In such a case the vehicle would skid and not overturn if the value of 'f' is less than b/2h. On the other hand the vehicle would overturn on the outer side before skidding if the value of b/2h is lower than 'f'. Thus the relative danger of lateral skidding and overturning depends on whether f is lower or higher than b/2h.

If the pavement is kept horizontal across the alignment, the pressure on the outer wheels will be higher due to the centrifugal force acting outwards and hence the reaction R_B at the outer wheel would be higher. The difference in pressure distribution at inner and outer wheels has been indicated in Fig. 4.19. When the limiting equilibrium condition for overturning occurs the pressure at the inner wheels becomes equal to zero.

4.4.4 Superelevation

In order to counteract the effect of centrifugal force and to reduce the tendency of the vehicle to overturn or skid, the outer edge of the pavement is raised with respect to the inner edge, thus providing a transverse slope throughout the length of the horizontal curve. This transverse inclination to the pavement surface is known as superelevation or cant or banking. The superelevation 'e' is expressed as the ratio of the height of outer edge with respect to the horizontal width. From Fig. 4.20 it may be seen that superelevation,

$$e = \frac{NL}{ML} = \tan\theta$$

In practice the inclination θ with the horizontal is very small and the value of tan θ seldom exceeds 0.07. Therefore the value of than θ is practically equal to sin θ.

Hence, $e = \tan\theta \approx \sin\theta = \frac{E}{B}$ which is measured as the ratio of the relative elevation of the outer edge, E to width of pavement, B. This is more convenient to measure.

If e is the superelevation rate and E is the total superelevated height of outer edge, the total rise in outer edge of the pavement with respect to the inner edge = NL = E = eB.

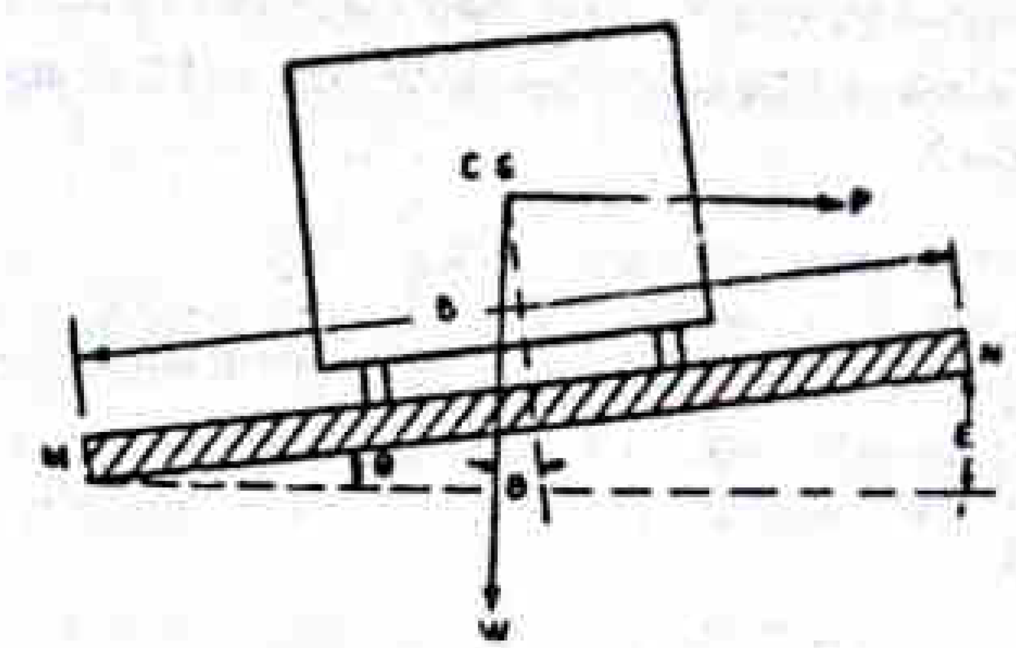


Fig. 4.20 Superelevated Pavement Section

Analysis of superelevation

The forces acting on the vehicle while moving on a circular curve of radius R metres at speed of v m/sec are

(i) the centrifugal force $P = Wv^2/gR$ acting horizontally outwards through the center of gravity, CG

(ii) the weight W of the vehicle acting vertically downwards through the CG

(iii) the frictional force developed between the wheels and the pavement counteractions transversely along the pavement surface towards the center of the curve.

The centrifugal force is thus opposed by corresponding value of the friction developed and by a component of the force of gravity due to the superelevation provided. Figure 4.21 shows the cross section of a pavement with all the forces acting on the vehicle resolved parallel and perpendicular to the inclined road surface. Considering the equilibrium of the components of forces acting parallel to the plane, (P cos θ) the component of centrifugal force is opposed by (W sin θ) the component of gravity and the frictional forces F_A and F_B.

For equilibrium condition,

$$P\cos\theta = W\sin\theta + F_A + F_B$$

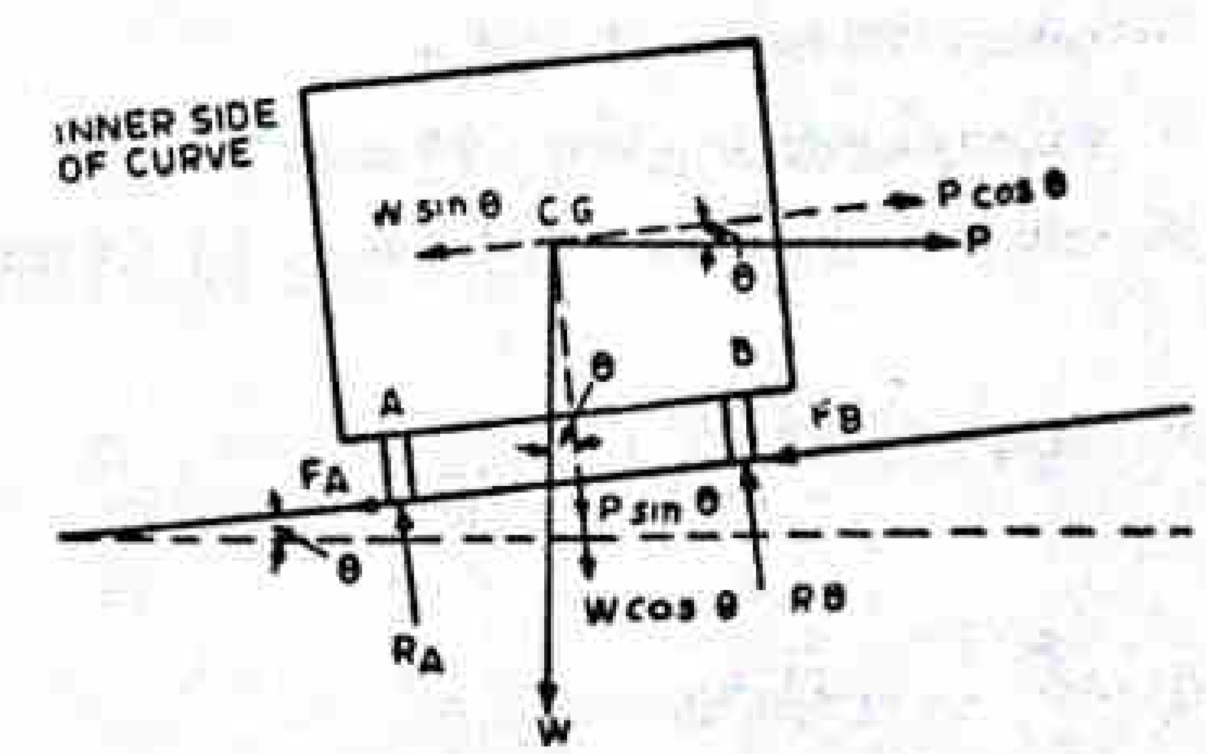


Fig. 4.21 Analysis of Superelevation

The limiting equilibrium is reached when the full values of the frictional forces are developed and the values of F_A and F_B reach their maximum value of $f \times R_B$ and $f \times R_A$ respectively where 'f' is the coefficient of lateral friction and R_A and R_B are the normal reactions at wheels A and B.

Therefore,

$$P \cos\theta = W \sin\theta + f(R_A + R_B)$$

$$= W \sin\theta + f(W \cos\theta + P \sin\theta)$$

i.e.,

$$P(\cos\theta - f \sin\theta) = W \sin\theta + f W \cos\theta$$

Dividing by $W \cos\theta$,

$$\frac{P}{W}(1 - f \tan\theta) = \tan\theta + f$$

$$\frac{P}{W} = \frac{\tan\theta + f}{1 - f \tan\theta}$$

The value of coefficient of lateral friction, 'f' is taken as 0.15 for design purposes, (See article 4.1.1). The value of $\tan\theta$ or transverse slope due to superelevation seldom exceeds 0.07 or about 1/15. Hence the value of $f \tan\theta$ is about 0.01. Thus the value of $(1 - f \tan\theta)$ in the above equation is equal to 0.99 and may be approximated to 1.0.

Therefore,

$$\frac{P}{W} \approx \tan\theta + f = e + f$$

But

$$\frac{P}{W} = \frac{v^2}{gR}$$

Therefore,

$$e + f = \frac{v^2}{gR} \quad (4.7)$$

Here
- e = rate of superelevation = $\tan\theta$
- f = design value of lateral friction coefficient = 0.15
- v = speed of the vehicle, m/sec
- R = radius of the horizontal curve, m
- g = acceleration due to gravity = 9.8 m/sec^2

If the speed of the vehicle is represented as V kmph, the Eq. 4.8 may be written as follows ;

$$E + f = \frac{(0.278V)^2}{9.8R} = \frac{V^2}{127R}$$

i.e.

$$e + f = \frac{V^2}{127R} \quad (4.8)$$

- V = speed, kmph
- R = radius, surfaces

If the coefficient of friction is neglected or assumed equal to zero, i.e. if f = 0, the *equilibrium superelevation* required to counteract the centrifugal force fully will be given by :

$$e = \frac{v^2}{gR} \quad \frac{V^2}{127R}$$

If superelevation is provided according to this formula, the pressures on the outer and inner wheels will be equal; but this will result in a very high value of superelevation. As considerable role is played by the lateral frictional resistance in counteracting the centrifugal force, it is always taken into account. In places where superelevation is not provided due to practical difficulties, i.e. where e = 0 and $f = \frac{v^2}{gR} = \frac{V^2}{127R}$, and the frictional force has to fully counteract the centrifugal ratio. In some types of intersections it is not possible to provide superelevation and in such cases the friction counteracts the centrifugal force fully; with no superelevation, the allowable speed of vehicle negotiating, a turn should be restricted to the condition,

$$f = \frac{v^2}{gR} = \frac{V^2}{127R}, \text{ or } V = \sqrt{127 f R}$$

It is possible that at some intersections, a negative superelevation is unavoidable.

Thus the superelevation 'e' required on a horizontal curve depends on the radius of the curve R, speed of the vehicle V and the coefficient of lateral friction or the transverse skid resistance f. Therefore, in order to assess the superelevation e required, the speed is taken as equal to the design speed of the road and the minimum value of transverse skid resistance f for design purpose is standardised equal to 0.15.

Example 4.8

The radius of a horizontal circular curve is 100 m. The design speed is 50 kmph and the design coefficient of lateral friction is 0.15.

(a) calculate the superelevation required if full lateral friction is assumed to develop.

(b) calculate the coefficient of friction needed if no superelevation is provided.

(c) calculate the equilibrium superelevation if the pressure on inner and outer wheels should be equal.

Solution

(a) Superelevation is given by the relation

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R} \quad \text{(Equation 4.7 \& 4.8)}$$

Here

$$f = 0.15; V = 50 \text{ kmph or } v = \frac{50}{3.6} \text{ m/sec.}$$

$$R = 100 \text{ m}$$

$$e + 0.15 = \frac{50^2}{127 \times 100} = 0.917$$

$$e = 0.197 - 0.15 = 0.047$$

i.e., superelevation rate is 1 in 21.2

(b) If no superelevation is provided, e = 0 and friction factor developed,

$$f = \frac{V^2}{127R} = \frac{50^2}{127 \times 100} = 0.917$$

(c) For the pressure on inner and outer wheels to be equal or for equilibrium superelevation counteracting centrifugal force fully, f = 0 and

$$e = \frac{V^2}{127R} = \frac{50^2}{127 \times 100} = 0.917$$

i.e., equilibrium superelevation rate is 1 in 5.1. However this rate of superelevation being very high, cannot be provided.

Maximum superelevation

As per Equation 4.7 and 4.8, the value of superelevation needed increases with increase in speed and with decrease in radius of the curve, for a constant value of coefficient of lateral friction 'f'. From the practical view point it will be necessary to limit the maximum allowable superelevation to avoid very high values of 'e'. This is particularly necessary when the road has to cater for mixed traffic, consisting of fast and slow traffic.

In the case of heavily loaded bullock carts and trucks carrying less dense materials like straw or cotton, the centre of gravity of the loaded vehicle will be relatively high and it will not be safe for such vehicles to move on a road with a high rate of superelevation. Because of the slow speed, the centrifugal force will be negligibly small in the case of bullock carts. Hence to avoid the danger of toppling of such loaded slow moving vehicles, it is essential to limit the value of maximum allowable superelevation. Indian Roads Congress had fixed the maximum limit of superelevation in plain and rolling terrains and is snow bound areas as 7.0 percent taking such mixed traffic into consideration. However, on hill roads not bound by snow a maximum superelevation upto 10 percent has been recommended. On urban road stretches with frequent intersections, it may be necessary to limit the maximum superelevations to 4.0 percent, keeping in view the convenience in construction and that of turning movements of vehicles.

Minimum Superelevation

From drainage considerations it is necessary to have a minimum cross slope to drain off the surface water. If the calculated superelevation from Equation 4.8 worksout to be equal to or less than the camber of the road surface, then the minimum superelevation to be provided on horizontal curve may be limited to the camber of the surface. Thus after the elimination of the crown a uniform cross slope equal to the camber is maintained from outer to inner edge of pavement at the circular curve. In very flat curves with large radius

the centrifugal force developed will be very small and in such cases the normal camber may be retained on the curves. Though this practice will cause a negative superelevation on the outer half of the pavement due to the normal camber, the centrifugal force together with this negative superelevation would be considerably less than the allowable friction coefficient on such curves. The IRC recommendation giving the radii of horizontal curves beyond which normal cambered section may be maintained and no superelevation is required for curves, are presented in Table 4.9, for various design speeds and rates of cross slope.

Table 4.9 Radii beyond which Superelevation is not required

Design speed (kmph)	Radius (metre) of horizontal curve for camber of :				
	4 %	3 %	2.5 %	2 %	1.7 %
20	50	60	70	90	100
25	70	90	110	140	150
30	100	130	160	200	240
35	140	180	220	270	320
40	180	240	280	350	420
50	280	370	450	550	650
60	470	620	750	950	1100
80	700	950	1100	1400	1700
100	1100	1500	1800	2200	2600

Superelevation Design

Design of superelevation for mixed traffic conditions is complex problem, as different vehicles ply on the road with a wide range of speeds. To superelevate the pavement upto the maximum limit so as to counteract the centrifugal force fully, neglecting the lateral friction is safer for fact moving vehicles. But for slow moving vehicles this may quite inconvenient. On the contrary to provide lower value of superelevation thus relying more on the lateral friction would be unsafe for fast moving vehicles. As a compromise and from practical considerations it is suggested that the superelevation should be provided to fully counteract the centrifugal force due to 75 percent of the design speed, (by neglecting lateral friction developed) and limiting the maximum superelevation to 0.07 (except on hill roads, not bound by snow where the maximum allowable value is 0.1).

Steps for superelevation design :

Various steps in the design of superelevation in practice may be summarized as given below :

Step (i) The superelevation for 75 percent of design speed (v m/sec or V kmph) is calculated neglecting the friction

$$e = \frac{(0.75\,v)^2}{gR} \text{ or } \frac{(0.75\,V)^2}{127\,R}$$

i.e.,

$$e \approx \frac{V^2}{225\,R} \tag{4.9}$$

Step (ii) If the calculated value of 'e' is less than 7% or 0.07 the value so obtained is provided. If the value of 'e' as per equation 4.9 exceeds 0.07 then provide the maximum superelevation equal to 0.07 and proceed with steps (iii) or (iv).

Step (iii) Check the coefficient of friction developed for the maximum value of e = 0.07 at the full value of design speed,

$$F = \left(\frac{v^2}{gR} - 0.07\right) \quad (4.10)$$

$$= \left(\frac{v^2}{127\,R} - 0.07\right)$$

If the value of f thus calculated is less than 0.15, the superelevation of 0.07 is safe for the design speed. If not, calculate the restricted speed as given in step (iv).

Step (iv) As an alternative to step (iii), the allowable speed (v_a m/sec. or V_a kmph) at the curve is calculated by considering the design coefficient of lateral friction and the maximum superelevation, i.e.,

$$e + f = 0.07 + 0.15$$

$$= 0.22 = \frac{v_a^2}{gR} = \frac{V_a^2}{127\,R}$$

calculate the safe allowable speed,

$$v_a = \sqrt{0.22\,gR} = \sqrt{2.156\,R} \text{ m/sec}$$

or

$$V_a = \sqrt{27.94\,R} \text{ kmph} \quad (4.11)$$

If the allowable speed, as calculated above is higher than the design speed, then the design is adequate and provides a superelevation of 'e' equal to 0.07. If the allowable speed is less than the design speed, the speed is limited to the allowable speed V_a kmph calculated above.

Appropriate warning sign and speed limit regulation sign are installed to restrict and regulate the speed at such curves when the safe speed V_a is less than the design speed V. For important highways, it is desirable to design the road without speed restriction at curves, as far as possible. Hence if site conditions permit, the curve should be re-aligned with a larger radius of curvature so that the design speed could be maintained (See Art. 4.4.4 and Table 4.10 for radius of horizontal curve).

Example 4.9

A two lane road with design speed 80 kmph has horizontal curve of radius 480 m. Design the rate of superelevation for mixed traffic. By how much should the outer edges of the pavement be raised with respect to the centre line, if the pavement is rotated with respect to the centre line and the width of the pavement at the horizontal curve is 7.5 m.

Solution

For mixed traffic conditions the superelevation should fully counteract the centrifugal force for 75% of design speed.

Hence using Equation 4.9.

$$e = \frac{V^2}{225\,R} = \frac{80^2}{225 \times 480} = 0.059$$

Since this value is less than 0.07, the superelevation of 0.059 may be adopted.

The total width of pavement B = 7.5 m.

Raising of outer edge with respect to centre

$$= E = \frac{B.e}{2} = \frac{7.5}{2} \times 0.059 = 0.22 \text{ m}$$

Example 4.10

Design the rate of superelevation for a horizontal highway curve of radius 500 m and speed 100 kmph.

Solution

For mixed traffic conditions, superelevation is given by Eq. 4.9.

$$e = \frac{V^2}{225\,R}$$

$$V = 100 \text{ kmph}$$

$$R = 500 \text{ m}$$

$$e = \frac{100^2}{225 \times 500} = 0.089$$

As the value is greater than the maximum superelevation of 0.07, the actual superelevation to be provided is restricted to 0.07.

Check for coefficient of lateral friction developed for full speed using Eq. 4.10.

$$f = \frac{V^2}{127\,R} - 0.07 = \frac{100^2}{127 \times 500} - 0.07$$

$$= 0.157 - 0.07 = 0.087$$

As the value is less than 0.15, the design is safe with a superelevation of 0.07.

Example 4.11

The design speed of a highway is 80 kmph. There is a horizontal curve of radius 200 m on a certain locality. Calculate the superelevation needed to maintain this speed. If the maximum superelevation of 0.07 is not to be exceeded, calculate the maximum allowable speed on this horizontal curve as it is not possible to increase the radius. Safe limit of transverse coefficient of friction is 0.15.

Solution

The problem may be solved by considering 75 percent design speed for finding the superelevation or counteract the centrifugal force fully using Eq. 4.9.

$$e = \frac{V^2}{225\,R}$$

i.e.,

$$e = \frac{80^2}{225 \times 200} = 0.142$$

Maximum allowable value of e is to be limited to 0.07.

Check for the value of friction developed,

$$f = \frac{V^2}{127\,R} - 0.07$$

$$= \frac{80^2}{127 \times 200} - 0.07 = 0.18$$

As this value is greater than the maximum allowable safe friction coefficient of 0.15 and also as the radius can not be increased, the speed has to be restricted.

Hence the maximum allowable speed (V_a kmph) on this curve is obtained by assuming the full value of design friction coefficient on 0.15. This is given by the Eq. 4.11.

$$V_a = \sqrt{27.94R} = 74.75 \text{ kmph}$$

Hence the speed may be restricted to less than 74 or say 70 kmph at this curve.

Example 4.12

A major District Road with thin bituminous pavement surface in low rainfall area has horizontal curve of radius 1400 m. If the design speed is 65 kmph, what should be the superelevation ? Discuss.

Solution

Using Eq. 4.9,

$$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 1400} = 0.0134$$

The superelevation value required is only 0.0134 which is even less than the normal cross slope required to drain off the surface water. The recommended camber for thin bituminous pavement in low rainfall area (Table 4.1) is 2% or 0.02. The radius beyond which no superelevation is required for a speed of 65 kmph and 2% camber is 950 m as per the IRC (See table 4.9). As the radius of the horizontal curve in this case is 1400 m, there is no necessity of providing superelevation; therefore the normal camber of 2% may be retained at the horizontal curve.

However, check for safety against centrifugal force at design speed along with the negative superelevation at the outer half of the pavement due to the normal camber.

$$\text{Net transverse skid resistance} = -e + f = -0.02 + 0.15 = 0.13$$

$$\text{Centrifugal ratio} = \frac{V^2}{127\,R} = \frac{65^2}{127 \times 1400} = 0.024$$

As this value of 0.024 is considerably lower than the net transverse skid resistance of 0.13 available at the curve, this horizontal curve with normal cambered section is quite safe for a design speed of 65 kmph.

Attainment of Superelevation

Introducing superelevation on a horizontal curve in the field is an important feature in construction. The road cross section at the straight portion is cambered with the crown at the centre of the pavement and sloping down towards the edges. But the cross section in the circular curve portion of the road is superelevated with a uniform tilt sloping down from the outer edge of the pavement up to inner edge. These may be seen from *sections* at A and E of Fig. 4.24. Thus the crowned camber sections at the straight before the start of the transition curve should be changed to a single cross slope equal to the desired superelevation at the beginning of the circular curve. This change may be conveniently attained at a gradual and uniform rate throughout the transition length of the horizontal curve. The full superelevation is attained by the end of transition curve or at the beginning of the circular curve.

The attainment of superelevation may be split up into two parts :

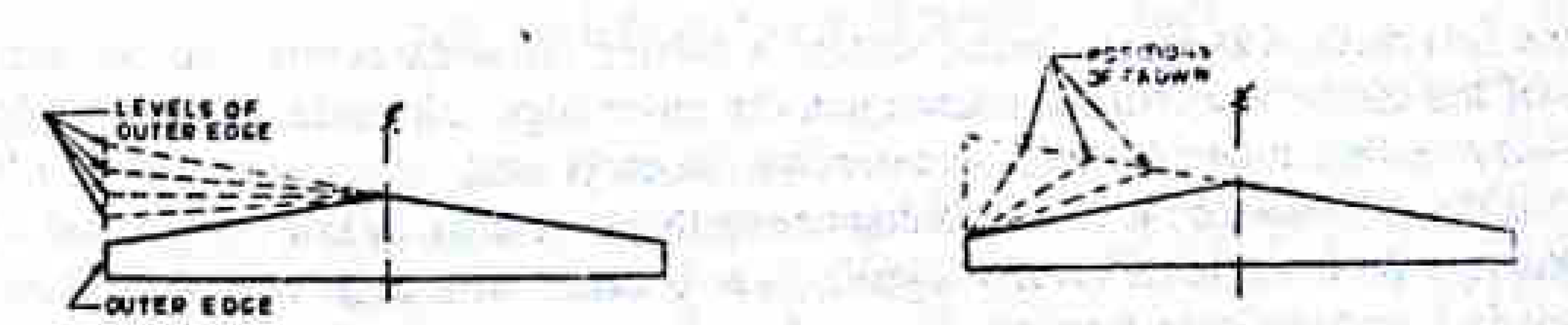
(a) Elimination of crown of the cambered section

(b) Rotation of pavement to attain full superelevation

Elimination of crown of the cambered section

This may be done by two methods. In the first method, the outer half of the cross slope is rotated about the crown at a desired rate such that the surface falls on the same plane as the inner half and the elevation of the centre line is not altered. (Ref. Fig. 4.22a).

The outer half of the cross slope is brought to level or horizontal (by rotating about the crown line) at the start of the transition curve or at tangent point T.P. See cross section at B in Fig. 4.24. Subsequently the outer half is further rotated so as to obtain uniform cross slope equal to the camber, as shown in Fig. 4.22 (a) and in cross section C of Fig. 4.24.



(a) Outer edge rotated about the crown *(b) Crown shifted outwards (diagonal crown method)*

Fig. 4.22 Elimination of Crown of Cambered Section

Thus no point on the curve will have a negative superelevation at the outer half of the pavement event at the start of the transition curve. This method has a drawback that the surface drainage will not be proper at the outer half, during a short stretch of the road with a cross slope less than the camber between point A and C in Fig. 4.24.

In the second method of eliminating the crown, known as diagonal crown method, the crown is progressively shifted outwards, thus increasing the width of the inner half of cross section progressively. This method is not usually adopted as a portion of the outer half of the pavement has increasing values of negative superelevation on to a portion of the outer half, before the crown is eliminated (see Fig. 4.22 b).

Rotation of pavement to attain full superelevation

When the crown of the camber is eliminated, the superelevation available at this section is equal to the camber. But the superelevation to be provided at the beginning of circular curve may be greater than the camber in many cases when the design superelevation is more than the minimum. Hence the pavement section will have to be rotated further till the desired banking is obtained.

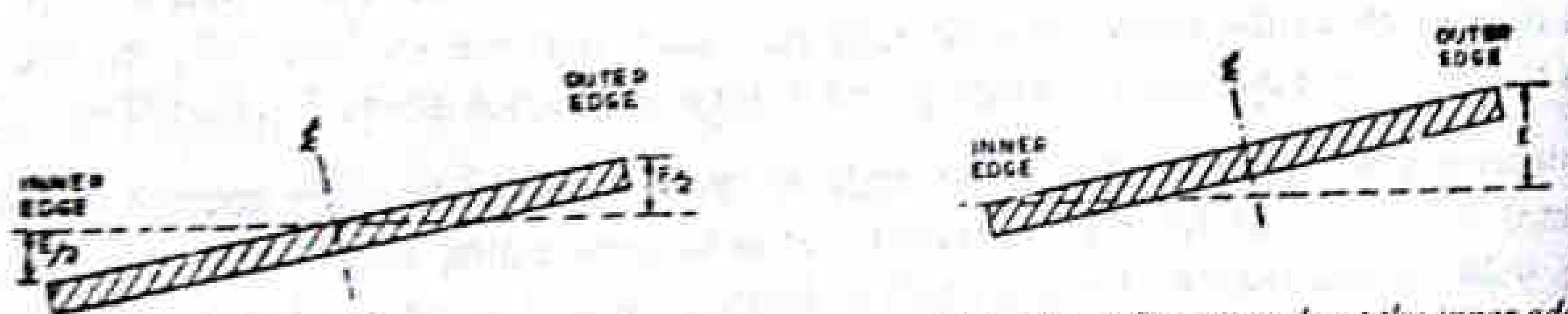
As an example, if the specified camber in a bituminous pavement surface is 0.02 and the design superelevation is 0.07, the camber is first eliminated resulting in a superelevation of 0.02 and then the cross slope is further increased till it attains the full superelevation of 0.07. If the designed superelevation is 'e' and the total width of the pavement at the horizontal curve is 'B', the total banking of the outer edge of the pavement with respect to the inner edge is equal to E = B.e.

There are two methods of rotating the pavement cross section to attain the full superelevation after the elimination of the camber.

(i) By rotating the pavement cross section about the centre line, depressing the inner edge and raising the outer edge each by half the total amount of superelevation, i.e. by E/2 with respect to the centre.

(ii) By rotating the pavement cross section about the inner edge of the pavement section raising both the centre as well as the outer edge of the pavement such that the outer edge is raised by the full amount of superelevation, E with respect to the inner edge.

The two methods are shown in Fig. 4.23.



Method (i) Rotating about centre line *Method (ii) Rotating about the inner edge*

Fig. 4.23 Rotation of Pavement Section to attain Full Superelevation

In the first method as the pavement section is rotated about the centre line, the vertical profile of the centre line remains unchanged; the outer edge is banked and inner edge is depressed resulting in an advantage in balancing the earth work. The disadvantage of this method is the drainage problem due to depressing the inner edge below the general level. The drainage problem is of greater significance in areas with high rain fall when the subgrade is in cutting or in level terrain. If the subgrade is in embankment or when the road has a significant gradient to facilitate longitudinal drainage, there will be no drainage problem.

The second method of rotating about the inner edge is preferably in very flat terrain in high rain fall areas, when the road is not taken on embankment, in order to avoid the drainage problem. But the entire pavement width and outer shoulder should also be raised with respect to the inner edge by additional earth fill. In this case the centre of the pavement is also raised, which may be considered as a disadvantage of the method as the vertical alignment of the road is altered.

The attainment of superelevation has been shown in detail in Fig. 4.24. The plan of the horizontal curve including the straight, transition and circular curves is shown in

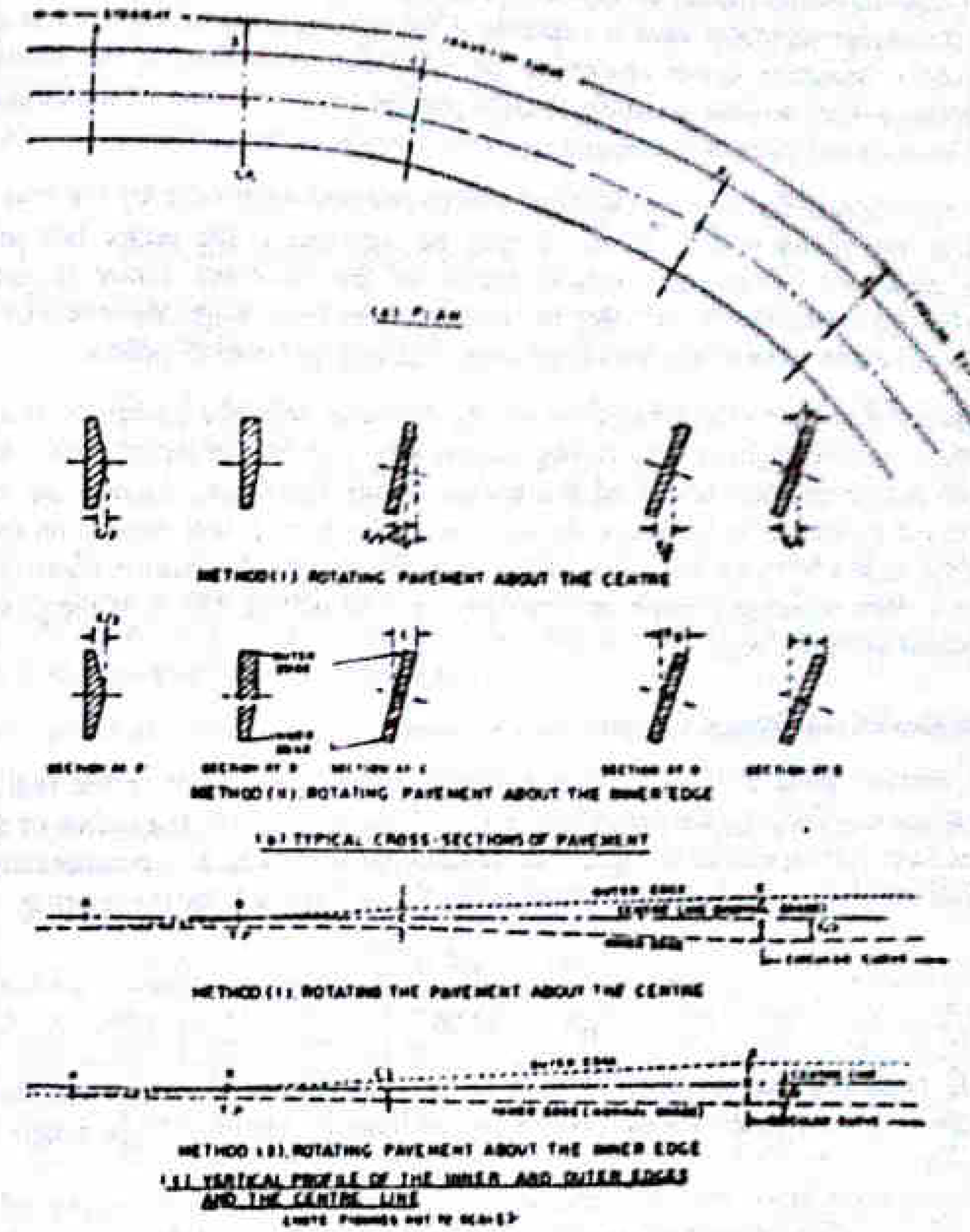


Fig. 4.24 Attainment of Superelevation

Fig. 4.24 a. Elimination of the crown of cambered section, attainment of uniform slope and the two methods of rotating the pavement section to attain full superelevation have been illustrated in Fig. 4.24b. The outer half of the cambered section is raised to a horizontal position between A and B at the same rate of introduction of superelevation along the transition curve of length L_s. Thus at the tangent point B there is no negative superelevation.

When the pavement is rotated about the inner edge, the length AB is given by :

$$\frac{c\,B\,N}{2} = \frac{c\,L_s}{2\,e}$$

where c and e are the rates of camber and superelevation, B is the width of pavement and N is the rate of raising the outer edge of pavement along the transition curve of length L_s. At point C the pavement attains uniform cross slope equal to the camber and the distance BC = AB. The pavement is further rotated at the same rate between C and E to attain full superelevation.

The superelevation should be attained gradually over the full length of transition curve so that the design superelevation is available at the starting point of the circular curve. In cases where transition curve cannot be provided for some reason, two-thirds of the superelevation may be attained at the straight portion before the start of the circular curve and the balance one-third at the beginning of the circular curve.

The vertical profiles of the inner edge, centre line and outer edge by the two methods of rotation are shown in Fig. 4.24c. It may be seen that in the centre line method of rotating pavement section the vertical profile of the pavement centre is not altered throughout the horizontal curve. But by rotating about inner edge, the levels of both the centre line and that outer edges are raised above the original vertical profile.

The superelevation is introduced by raising the outer edge the pavement at a rate not exceeding 1 to 150 in plain and rolling terrain and 1 in 60 on mountainous and steep terrain as per recommendations of the Indian Roads Congress. Hence the length of transition curve needed to introduce the total superelevation E will depend on the rate of introducing superelevation and value of E. Thus the length of transition curve needed to introduce a total superelevation E at a rate of 1 in 150 will be 150 E, if the pavement is rotated about the inner edge.

4.4.5 Radius of Horizontal Curve

For a certain speed of vehicle the centrifugal force is dependent on the radius of the horizontal curve. To keep the centrifugal ratio within a low limit, the radius of the curve should be kept correspondingly high. The centrifugal force which is counteracted by the superelevation and lateral friction is given as per Eq. 4.7 and 4.8, by the relation.

$$e + f = \frac{v^2}{gR} = \frac{V^2}{127R}$$

In this equation, the maximum allowable superelevation rate has been fixed as 7 percent or 0.07 and the design coefficient of lateral friction 'f' is taken as 0.15 (Art. 4.1.2).

Hence, $$e + f = 0.07 + 0.15 = 0.22 = \frac{v^2}{gR} = \frac{V^2}{127R}$$

If the design speed is decided for a highway, then the minimum radius to be adopted can be found from the above relationship.

Thus the ruling minimum radius of the curve for ruling design speed v m/sec. or V/kmph is given by :

$$R_{ruling} = \frac{v^2}{(e+f)\,g} \quad (4.12)$$

Also, $$R_{ruling} = \frac{V^2}{127\,(e+f)} \quad (4.13)$$

When the minimum design speed V' kmph is adopted (see Table 4.8) instead of ruling design speed V kmph, the absolute minimum radius of horizontal curve R_{min} is given by :

$$R_{min} = \frac{V'^2}{127\,(e+f)} \quad (4.14)$$

In the above equations,

- v and V = ruling design speeds, in m/sec and kmph respectively
- V' = minimum design speed, kmph
- E = rate of superelevation; the maximum value of e is taken as 0.07 at all the regions except at hill roads without snow where it is taken as 0.1.
- f = design value of transverse skid resistance or coefficient of friction, taken as 0.15
- g = acceleration due to gravity = 9.8 m/sec^2

According to the earlier specifications of the IRC, the ruling minimum radius of the horizontal curve was calculated from a speed value, 16 kmph higher than the design speed i.e., (V + 16) kmph. However now the calculations are based on the ruling and minimum design speeds given in Table 4.8.

The ruling and absolute minimum values of radii of horizontal curve of various classes of roads in different terrains (as per the latest IRC specifications) are given in Table 4.10.

Table 4.10 Minimum radii of horizontal curves for different terrain conditions, m

Classification of roads	Plain terrain		Rolling terrain		Mountainous terrain: Area not affected by snow		Mountainous terrain: Snow bound areas		Steep terrain: Area not affected by snow		Steep terrain: Snow bound areas	
	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini.	Absolute Mini	Rulling Mini	Absolute Mini
NH& SH	360	230	230	155	80	50	90	60	50	30	60	33
MDR	230	155	155	90	50	30	60	33	30	14	33	15
ODR	155	90	90	60	30	20	33	23	20	14	23	15
VR	90	60	60	45	20	14	23	15	20	14	23	15

Note : The values of ruling minimum and absolute minimum radii correspond to the ruling and minimum design speed values given in Table 4.8.

Example 4.13

Calculate the values of ruling minimum and absolute minimum radius of horizontal curve of a National Highway in plain terrain. Assume ruling design speed and minimum design speed values as 100 and 80 kmph respectively.

Solution

Ruling minimum radius is calculated using Eq. 4.12 or 4.13 for ruling design speed of 100 kmph with the maximum values of e = 0.07 and f = 0.15.

$$R_{ruling} = \frac{V^2}{127\,(e+f)} = \frac{100^2}{127\,(0.07+0.15)} = 357.9 \text{ m say } 360$$

The absolute minimum radius is calculated from the minimum design of V" = 80 kmph, using Eq. 4.14.

$$R_{min} = \frac{V'^2}{127\,(e+f)} = \frac{80^2}{127\,(0.07+0.15)} = 229.1 \text{ m say } 230 \text{ m}$$

Therefore provide rulling minimum radius of 360 m and absolute minimum radius of 230 m.

4.4.6 Widening of Pavement on Horizontal Curves

On horizontal curves, especially when they are not of very large radii, it is common to widen the pavement slightly more than the normal width. The object of providing extra widening of pavements on horizontal curves are due to the following reasons :

(a) An automobile has a rigid wheel base and only the front wheels can be turned; when this vehicle takes a turn to negotiate a horizontal curve, the rear wheels do not follow the same path as that of the front wheels. This phenomenon is called *off tracking*. Normally (at low speeds and up to the design speed when no lateral slipping of rear wheels take place) the rear wheels follow the inner path on the curve as compared with those of the corresponding front wheels. This means that if inner front wheel takes a path on the inner edge of a pavement at a horizontal curve, inner rear wheel will be off the pavement on the inner shoulder. The off-tracking depends on the length of the wheel base of the vehicle and the turning angle or the radius of the horizontal curve negotiated. This is illustrated in Fig. 4.25.

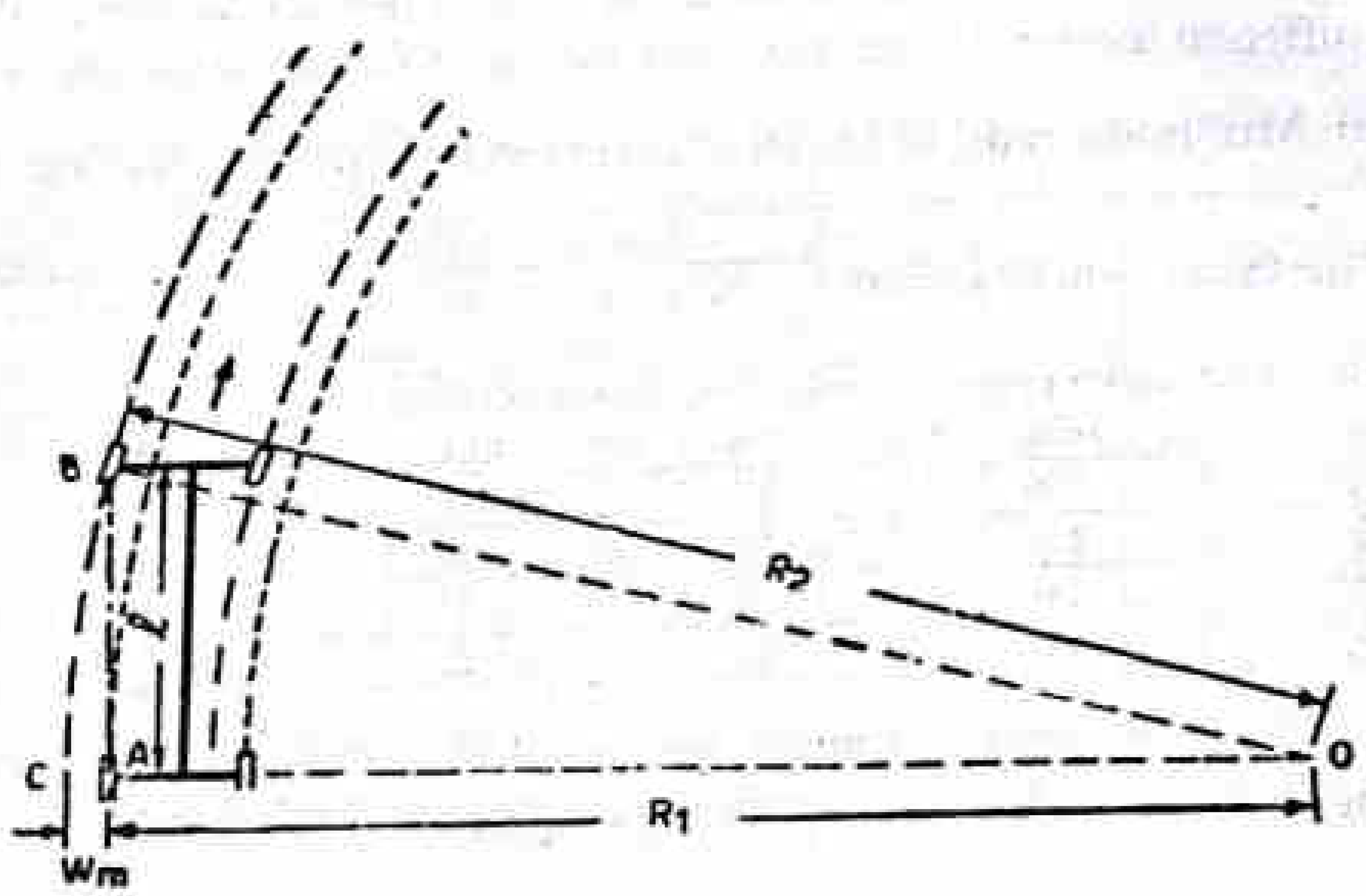


Fig. 4.25 Mechanical Widening on Horizontal Curve

(b) At speeds higher than the design speeds when the superelevation and lateral friction developed are not fully able to counteract the outwards thrust due to the centrifugal force, some transverse skidding may occur and the rear wheels may take paths on the outside of those traced by the front wheels on the horizontal curves. However this occurs only at excessively high speeds.

(c) The path traced by the wheels of a trailer in the case of trailer units, is also likely to be on either side of the central path of the towing vehicle, depending on the speed, rigidity of the universal joints and pavement roughness.

(d) In order to take curved path with larger radius and to have greater visibility at curve, the drivers have tendency not to follow the central path of the lane, but to use the outer side at the beginning of a curve.

(e) While two vehicles cross or overtake at horizontal curve there is a psychological tendency to maintain a greater clearance between the vehicles, than on straights for increase safety.

Thus the required extra widening of the pavement at the horizontal curves W_e depends on the length of wheel based of the vehicle l, radius of the curve negotiated R and the psychological factor which is a function of the speed of the vehicle and the radius of the curve.

It has been a practice therefore to provide extra width of pavement on horizontal curves when the radius is less than about 300 m.

Analysis of Extra Widening on Curves

The extra widening of pavement on horizontal curves is divided into two parts (i) mechanical and (ii) psychological widening.

Mechanical widening

The widening required to account for the off-tracking due to the rigidity of wheel based is called *mechanical widening* (W_m) and may be calculated as given below. Refer Fig. 4.25.

R_1 = radius of the path traversed by the outer rear wheel, m

R_2 = radius of the path traverse by the outer front wheel, m

W_m = off-tracking or the mechanical widening, m

l = length of wheel base, m

$$W_m = OC - OA = OB - OA = R_2 - R_1$$

From Δ OAB, $OA^2 = OB^2 - BA^2$

$$R_1^2 = R_2^2 - l^2$$

But $R_1 = R_2 - W_m$

$$(R_2 - W_m)^2 = R_2^2 - l^2$$

i.e., $R_2^2 - 2R_2W_m + W_m^2 = R_2^2 - l^2$

$$l^2 = W_m(2R_2 - W_m)$$

$$W_m = \frac{l^2}{2R_2 - W_m} \qquad (4.15)$$

$$= \frac{l^2}{2R} \text{ (approximately)}$$

Here R is the mean radius of the curve. The mechanical widening calculated above is required for one vehicle negotiating a horizontal curve along one traffic lane. Hence in a road having 'n' traffic lanes, as 'n' vehicles can travel simultaneously, the total mechanical widening required is given by

$$W_m = \frac{nl^2}{2R} \qquad (4.16)$$

Psychological widening

Extra width of pavement is also provided for psychological reasons such as, to provide for greater maneuverability of steering at higher speeds, to allow for the extra space requirements for the overhangs of vehicles and to provide greater clearance for crossing and overtaking vehicles on the curves. Psychological widening is therefore important in pavements with more than one lane. An empirical formula has been recommended by IRC for finding the additional psychological widening 'W_{ps}' which is dependent on the design speed V of the vehicle and the radius R of the curve. The psychological widening is given by the formula :

$$W_{ps} = \frac{V}{9.5\sqrt{R}} \qquad (4.17)$$

Hence the total widening We, m required on a horizontal curve is given by :

$$W_e = W_m + W_{ps}$$

i.e.,

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \qquad (4.18)$$

Hence n = number of traffic lanes.

l = length of wheel base of longest vehicle, m. The value of l may normally be taken as 6.1 m or 6.0 m for commercial vehicles, if not known.

V = design speed, kmph

R = radius of horizontal curve, m

The extra width recommended by the Indian Roads Congress for single and two lane pavements are given in Table 4.11.

Table 4.11 Extra width of pavement at horizontal curves

Radius of curve (m)	Upto 20	20 to 40	41 to 60	61 to 100	101 to 300	Above 300
Extra width (m)						
Two-lane	1.5	1.5	1.2	0.9	0.6	Nil
Single-lane	0.9	0.6	0.6	Nil	Nil	Nil

Note : For multi-lane roads, the pavement widening is calculated by adding half the extra width of two-lane roads to each lane of the multi-lane road.

Methods of introducing extra widening

The widening is introduced gradually, starting from the beginning of the transition curve or the tangent point (T.P.) and progressively increased at uniform rate, till the full value of designed widening 'W_e' is reached at the end of transition curve where full values of superelevation is also provided, as shown in Fig. 4.26. The full value of extra width W_e is continued throughout the length of the circular curve and then decreased gradually along the length of transition curve. Usually the widening is equally distributed i.e., $W_e/2$ each on inner and outer sides of the curve. But on sharp curves of hill roads the extra widening W_e may be provided in full on inside of the curve.

On horizontal circular curves without transition curves, two-thirds the widening is provided at the end of the straight section, i.e., before the start of the circular curve and the remaining one-third widening is provided on the circular curve beyond the tangent point as in the case of superelevation. In such cases, the widening is provided on the inside of the curve. Refer Fig. 4.27.

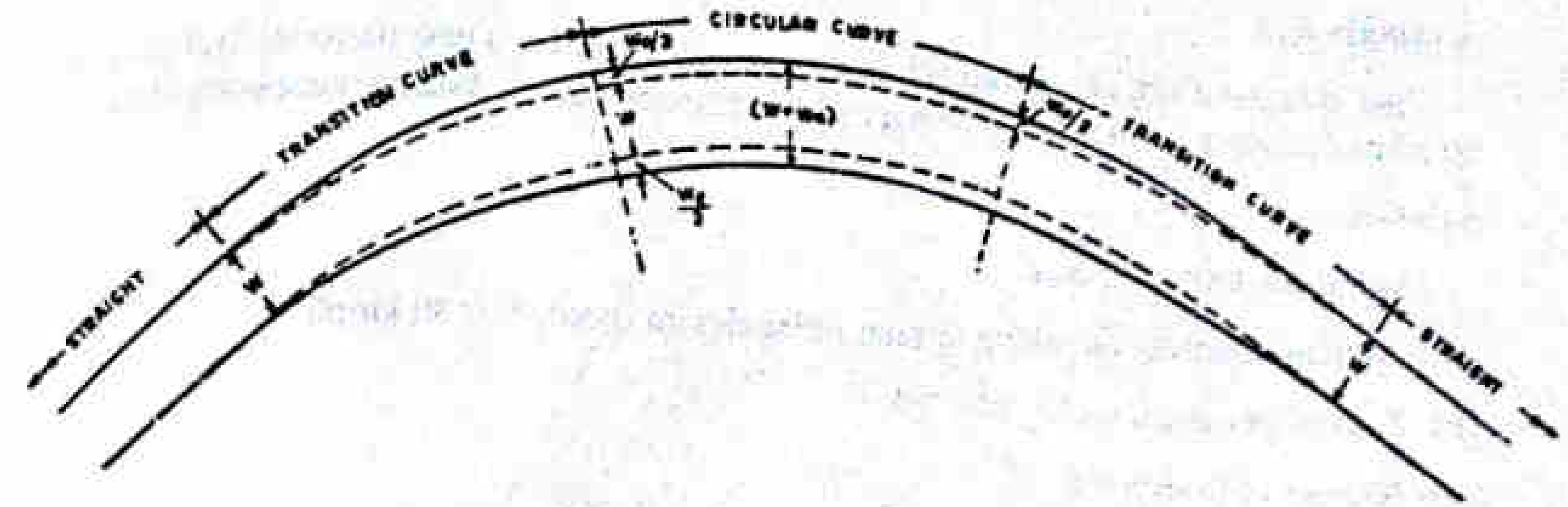


Fig. 4.26 Extra Widening of Pavement on Horizontal Curve

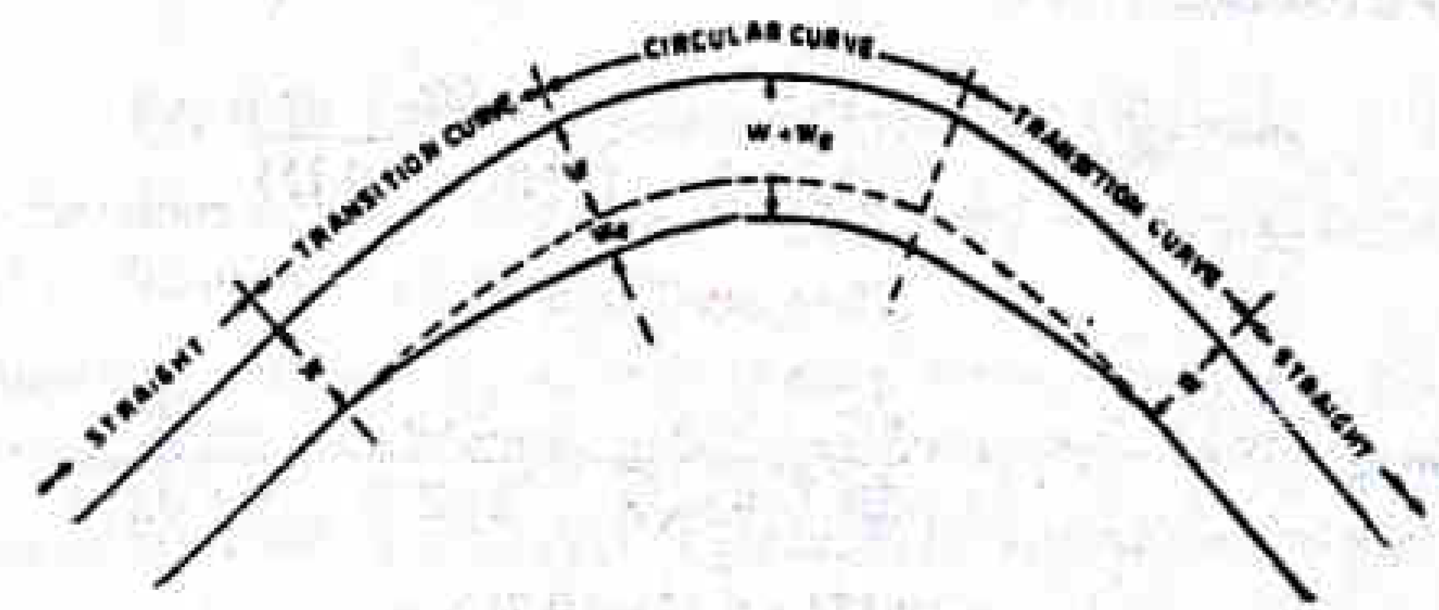


Fig. 4.27 Widening of Pavement on Sharp Curve

Example 4.14

Calculate the extra widening required for a pavement of within 7m on a horizontal curve of radius 250 m if the longest wheel base of vehicle expected on the road is 7.0 m. Design speed is 70 kmph. Compare the value obtained with IRC recommendations.

Solution

Extra widening required $W_e = W_m + W_{ps}$

$$= \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} \qquad (4.18)$$

Hence, n = 2 (two lanes for pavement width of 7.0 m)

l = 7.0

R = 250 m

V = 70 kmph

$$W_e = \frac{2\times7^2}{2\times250} + \frac{70}{9.5\sqrt{250}}$$

$$= 0.196 + 0.466 = 0.662 \text{ m}$$

The IRC recommends extra widening of 0.6 m when the radius of the curve is 101 to 300 m. (See Table 4.11)

Example 4.15

Find the total width of a pavement on a horizontal curve for a new national highway to be aligned along a rolling terrain with a ruling minimum radius. Assume necessary data.

Solution

Assume the following data :

(i) National highway on rolling terrain, ruling design speed, V = 80 kmph

(ii) Normal pavement width, W = 7.0

(iii) Number of lanes n = 2

(iv) Wheel base of the truck l = 6 m

(v) Maximum value of superelevation e = 0.07
and skid resistance f = 0.15

$$R_{ruling} = \frac{V^2}{127(e+f)} = \frac{80^2}{127(0.07+0.15)}$$

$$= 229 \text{ m, say } 230 \text{ m}$$

Extra widening $$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2\times6^2}{2\times230} + \frac{80}{9.5\sqrt{250}}$$

$$= 0.157 + 0.555 = 0.712 \text{ m}$$

Total pavement width on curve $= W + W_e = 7.0 + 0.71 = 7.71$ m

4.4.7 Horizontal Transition Curve

A transition curve has a radius which decreases from infinity at the tangent point to a designed radius of the circular curve. When a transition curve is introduced between a straight and circular curve, the radius of the transition curve decreases becomes minimum at the beginning of the circular curve. The rate of change of radius of the transition curve will depend on the equation of the curve or its shape.

Object of Providing Transition Curves

Suppose a curve of radius R takes off from straight road, and a vehicle travels on this road; then due to the centrifugal force which suddenly acts on the vehicle just after the tangent point, a sudden lateral jerk is felt on the vehicle. This not only causes discomfort to the passengers, but also makes it difficult to steer the vehicle safely. Refer Fig. 4.28. If a transition curve BC of length L_s is introduced between the straight AB and the circular curve CD of radius R, the centrifugal force will also be introduced gradually as the radius of the transition curve decreases gradually from infinity. The rate at which this force is introduced can be controlled by adopting suitable shape of the transition curve and by designing its length, so that the vehicle can have a smooth entry from the straight to the circular curve at the design speed.

A transition curve which is introduced between the straight and a circular curve will help also in gradually introducing the designed superelevation and the extra widening necessary.

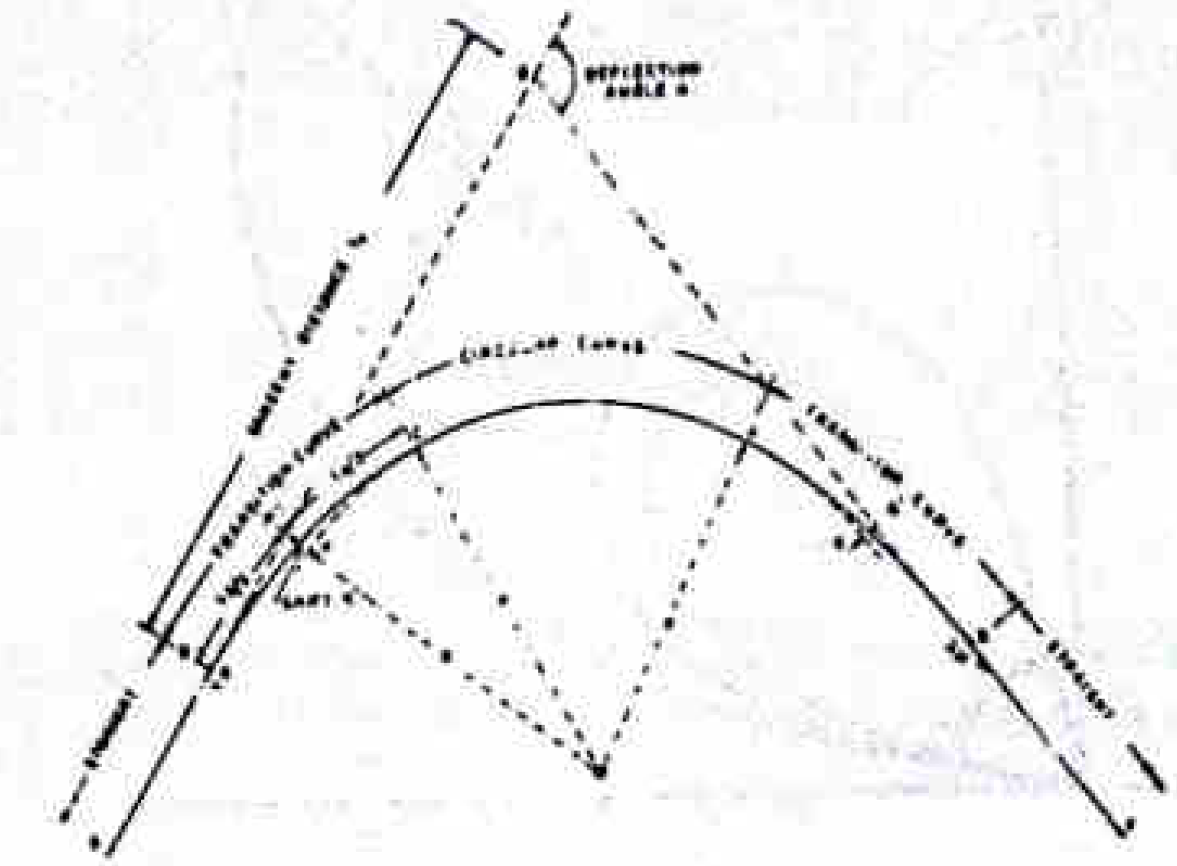


Fig. 4.28 Transition Curve in Horizontal Alignment

Thus the functions of transition curves in the horizontal alignment of highway may be summed up into the following points :

(a) to introduce gradually the centrifugal force between the tangent point and the beginning of the circular curve, avoiding a sudden jerk on the vehicle.

(b) to enable the driver turn the steering gradually for his own comfort and security.

(c) to enable gradual introduction of the designed superelevation and extra widening of pavement at the start of the circular curve.

(d) to improve the aesthetic appearance of the road.

In a good highway alignment it should be possible to maintain the design speed even on horizontal curves. The radius is first designed as discussed in article 4.3.4 and then a suitable shape of the transition curve is selected and its length is designed. *The ideal shape of a transition curve* should be such that the rate of introduction of centrifugal force or the rate of change of centrifugal acceleration should be consistent. This means that the radius of the transition curve should consistently decrease from infinity at the tangent point B (refer Fig. 4.28) to the radius R of the circular curve at point C, the end of the transition curve of length L_s. In an ideal transition curve the length L_s should be inversely proportional to the radius R i.e., ($L_s \propto 1/R$) or L_s R is a constant. The *spiral transition* fulfils this requirement.

Different Types of Transition Curves

The types of transition curves commonly adopted in horizontal alignment are :

(a) Spiral (also called *clothoid*)

(b) Lemniscate

(c) Cubic parabola

The general shapes of these three curves are shown in Fig. 4.29. All the three curves follow almost the same path upto deflection angle of 4°, and practically there is no significant difference even upto 9°. In all these curves, the radius decreases as the length

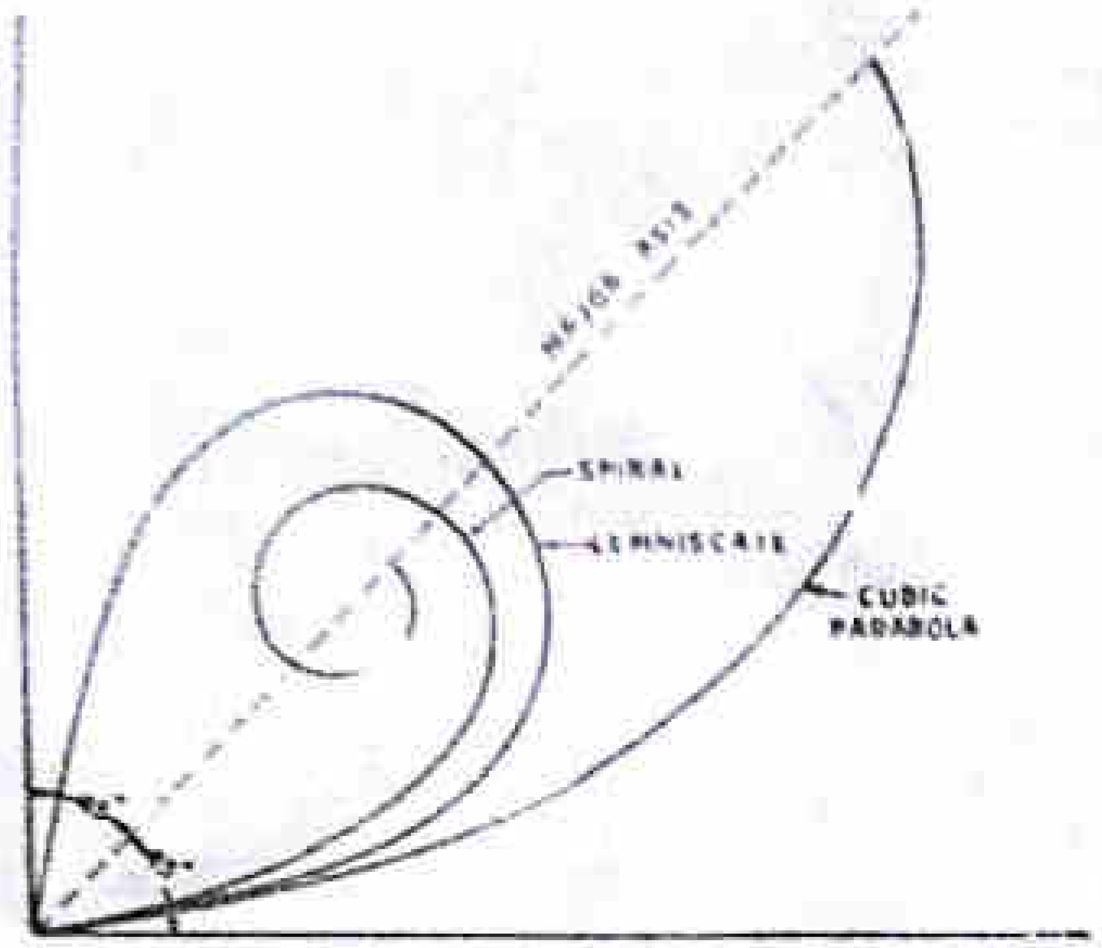


Fig. 4.29 Different Types of Transition Curves

increases. But the rate of change of radius and hence the rate of change of centrifugal acceleration is not constant in the case of lemniscate and cubic parabola, especially at deflection angles higher than 4°. In spiral curve the radius is inversely proportional to the length and the rate of change of centrifugal acceleration is uniform throughout the length of the curve. Thus the spiral fulfils the condition of an ideal transition curve.

The Indian Roads Congress recommends the use of the spiral as transition curve in the horizontal alignment of highways due to the following reasons :

(i) The spiral curve satisfies the requirements of an ideal transition.

(ii) The geometric property of spiral is such that the calculations and setting out the curve in the field is simple and easy.

The equation of the spiral may be written as :

$$L.R = L_S.R_C = \text{constant}$$

Therefore,

$$L = m\sqrt{\theta} \qquad (4.19)$$

Here m is a constant equal to $\sqrt{2RL_s}$ and θ is the tangent deflection angle in radius.

Calculation of Length of Transition Curve

The length of transition curve is designed to fulfil three conditions, viz. : (i) rate of change of centrifugal acceleration to be developed gradually (ii) rate of introduction of the designed superelevation to be at a reasonable rate (iii) minimum length by IRC empirical formula.

(i) *Rate of change of centrifugal acceleration* : At the tangent point the centrifugal acceleration (v^2/R) is zero at the radius R is infinity. At the end of the transition the radius R has the minimum value R_m. Hence the centrifugal acceleration is distributed over a length L_s of the transition curve. The centrifugal acceleration should be developed at such a low rate that it will not cause discomfort for the passengers of a vehicle traveling at the design speed (v m/sec). It is evident that larger the length of transition, lower will be the rate at which the centrifugal acceleration is introduced.

Let the length of transition curve be L_s metre. If 't' is the time taken in seconds to traverse this transition length at uniform design speed of v m/sec, $t = L_s/v$. The maximum centrifugal acceleration of v^2/R is introduced in time 't' through the transition length L_s and hence the rate of change of centrifugal acceleration 'C' is given by

$$C = \frac{v^2}{R.t} = \frac{v^2}{R\frac{L_s}{v}}$$

$$C = \frac{v^3}{L_s.R} \text{ (m/sec}^3\text{)} \qquad (4.20)$$

The maximum allowable value of the rate of change of centrifugal acceleration without producing discomfort or undesirable oscillation, is dependent on the speed and varies inversely with the radius. The IRC has recommended the following equation for finding the value of C for the design speed V kmph :

$$C = \frac{80}{(75+V)} \text{ m/sec}^3, [0.5 < C < 0.8] \qquad (4.21)$$

i.e., the minimum and maximum values of C are limited to 0.5 and 0.8 respectively.

Once the value of 'C' is decided based on the design speed as given above the length of transition curve L_s can be calculated from the Eq. 4.20 which may be rewritten as :

$$L_s = \frac{v^3}{CR} \qquad (4.22)$$

If the design speed is V kmph;

$$L_s = \frac{V^3}{(3.6)^3 CR}$$

i.e.,

$$L_s = \frac{V^3}{46.5CR} = \frac{0.0215V^3}{CR} \qquad (4.23)$$

Here,

L_s = length of transition curve, m

C = allowable rate of change of centrifugal acceleration, m/sec³ as given in Eq. 4.21.

R = radius of the circular curve, m.

(ii) *Rate of Introduction of superelevation* : In open country if a high value of superelevation is to be introduced, it is not desirable to raise the outer edge of a pavement at a larger rate than 1 in 150 relative to the grade of the centre line. Hence the length of transition curve should be atleast 150 times the total amount by which the outer edge of the pavement is to be raised with respect to the centre line. However, the transition curve length may be reduced by allowing an increased differential gradient of 1 in 100 in built up areas and 1 in 60 on hill roads. If the pavement is rotated about the inner edge and not the centre line, then the total lifting of outer edge with respect to inner edge has to be considered in calculating the length of transition curve required.

Let 'e' be the rate of superelevation designed as per Eq. 4.9 for a highway curve having normal pavement width W. Let 'W_e' be the extra widening provided at the circular curve so that the total width B of pavement = $(W + W_e)$ and the total raising of pavement with respect to the inner edge = e.B = e. $(W + W_e)$ = E. If it is assumed that the pavement is rotated about centre line after neutralizing the camber, (maintaining the vertical alignment of the centre line) then the maximum amount by which the outer edge is to be raised at the circular curve with respect to the centre = E/2. Hence allowing a rate of change of superelevation of 1 in N (where minimum value of N = 150 to 60 as discussed above), the length of transition curve L_S is given by :

$$L_S = \frac{EN}{2} = \frac{eN}{2}(W + W_e) \qquad (4.24a)$$

However if the pavement is rotated about the inner edge, the length of transition curve is given by :

$$L_S = EN = eN(W + W_e) \qquad (4.24b)$$

(iii) *By Empirical Formula* : According to the IRC standards, the length of horizontal transition curve L_S should not be less than the value given by the following equations for the terrain classifications :

(a) For plain and rolling terrain :

$$L_S = \frac{2.7V^2}{R} \qquad (4.25a)$$

(b) For mountainous and steep terrains;

$$L_S = \frac{V^2}{R} \qquad (4.25b)$$

The length of transition curve for the design should be the highest of the three values mentioned above. Therefore, the design steps are given below :

(a) Find the length of transition curve based on allowable rate of change of the centrifugal acceleration (Eq. 4.21 and 4.22 or 4.23).

(b) Find the length of transition curve based on rate of change of superelevation (Eq. 4.24 or 4.25).

(c) Check for the minimum required value of L_S as per Eq. 4.25a or 4.25b.

(d) Adopt the highest value of L_S given by (a), (b) and (c) above as the design length of transition curve.

The minimum length of transition curves for various values of radius of curve and design speeds recommended by the IRC for plain and rolling terrains and also for mountainous and steep terrains are given in Table 4.12.

Table 4.12 Minimum transition length for different speeds and curve radii

Plain and rolling terrain							Mountainous and steep terrain					
Curve Radius R(m)	Design speed (kmph)						Curve Radius R(m)	Design speed (kmph)				
	100	80	65	50	40	35		50	40	30	25	20
Transition length, m							Transition length, m					
45	-	-	-	-	NA	70	14	-	-	-	NA	30
60	-	-	-	NA	75	55	20	-	-	-	35	20
90	-	-	-	75	50	40	25	-	-	NA	25	20
100	-	-	NA	70	45	35	30	-	-	30	25	15
150	-	-	80	45	30	25	40	-	NA	25	20	15
170	-	-	70	40	25	20	50	-	40	20	15	15
200	-	NA	60	35	25	20	55	-	40	20	15	15
240	-	90	50	30	20	NR	70	NA	30	15	15	15
300	NA	75	40	25	NR	-	80	55	25	15	15	NR
360	130	60	35	20	-	-	90	45	25	15	15	-
400	115	55	30	20	-	-	100	45	20	15	15	-
500	95	45	25	NR	-	-	125	35	15	15	NR	-
600	80	35	20	-	-	-	150	30	15	15	-	-
700	70	35	20	-	-	-	170	25	15	NR	-	-
800	60	30	NR	-	-	-	200	20	15	-	-	-
900	55	30	-	-	-	-	250	15	15	-	-	-
1000	50	30	-	-	-	-	300	15	NR	-	-	-
1200	40	NR	-	-	-	-	400	15	-	-	-	-
1500	35	-	-	-	-	-	500	NR	-	-	-	-
1800	30	-	-	-	-	-						
2000	NR	-	-	-	-	-						

Note : NA – Not applicable; NR – Transition not required

The length of transition curve L_S required on a horizontal highway curve therefore depends upon the following factors :

(i) Radius of circular curve, R

(ii) Design speed, V

(iii) Allowable rate of change of centrifugal acceleration, C (which is also dependent on the design speed)

(iv) Maximum amount of superelevation, E which depends on the maximum rate of superelevation, *e* and the total width of the pavement, B at the horizontal curve

(v) Whether the pavement cross section is rotated about the inner edge or the centre line, after the elimination of the camber.

(vi) Allowable rate of introduction of superelevation, which depends on the terrain, location and environmental conditions of the site.

Setting out of transition curve

When transition curves are to be provided on both ends of a circular curve, the following procedure may be adopted. Refer Fig. 4.28. Let PCDQ be the original circular curve of radius R. PP' and QQ' are equal to the shift S of the transition curve given by the formula :

$$S = \frac{L_s^2}{24R} \qquad (4.26)$$

where L_s is the length of transition curve and R is the radius of the circular curve. B P C and E Q' D are the two transition curves, each of length L_s and C D is the shifted circular curve. The length of BP' and P'C are approximately equal to $L_s/2$. The points B and E remain as tangent points to the new compound curve B P' C D Q' E.

In order to set out the transition spiral, the design details such as the radius of the circular curve R, length of transition curve L_s, total deviation angle Δ, tangent deviation angle of the transition θ_s, central angle of circular arc Δ_c, tangent distance, apex distance etc. are determined. The curve may be laid either by off-set method or by polar deflection angle method. The details of calculating the off sets/deflection angles and setting out the curve in the field are not given here; they are available in the books so Surveying.

Example 4.16

Calculate the length of transition curve and the shift using the following data :

Design speed = 65 kmph

Radius of circular curve = 220 m

Allowable rate of introduction of superelevation (pavement rotated about the centre line) = 1 in 150

Pavement width including extra widening = 7.5 m

Solution

(a) Length of transition curve L_s as per allowable rate of centrifugal acceleration C :

Allowable rate of change of centrifugal acceleration as per Eq. 4.21,

$$C = \frac{80}{(75+V)} = \frac{80}{(75+65)} = 0.57, \text{ m/sec}^3$$

This value is between 0.5 and 0.8 and hence accepted.

$$L_s = \frac{0.0215\ V^3}{C\,R} = \frac{0.0215 \times 65^3}{0.57 \times 220} = 47.1 \text{ m}$$

(b) Length L_s by allowable rate of introduction of superelevation E :

Superelevation rate $$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 220} = 0.085$$

As this value is greater than the maximum allowable rate of 0.07, limit the value of e = 0.07. Check the safety against transverse skidding for the design speed of 65 kmph :

$$f = \frac{V^2}{127\,R} - e = \frac{65^2}{127 \times 220} - 0.07$$

$$= 0.15 - 0.07 = 0.08$$

As this value of f is less than the allowable value of 0.15, the superelevation rate of 0.07 is safe for the design speed of 65 kmph.

Total width of the pavement at the curve, B = 7.5 m

Total raise of outer edge of pavement with respect to the centre line

$$= \frac{E}{2} = \frac{e\,B}{2} = \frac{0.07 \times 7.5}{2} = 0.26 \text{ m}$$

Rate of introduction of superelevation, 1 in N = 1 in 150

$$L_s = \frac{E\,N}{2} = 0.26 \times 150 = 39 \text{ m}$$

(c) Minimum value of L_s as per IRC (Eq. 4.25)

$$= \frac{2.7\,V^2}{R} = \frac{2.7 \times 65^2}{220} = 51.9 \text{ m}$$

Adopt the highest value of the three i.e., 51.9 or say 52 m as the design length of transition curve.

$$\text{Shift } S = \frac{L_s^2}{24R} = \frac{52^2}{24 \times 220} = 0.51 \text{ m}$$

Example 4.17

A national Highway passing through rolling terrain in heavy rain fall area has a horizontal curve of radius 500 m. Design the length of transition curve assuming suitable data.

Solution

For a National Highway on rolling terrain, the following data may be assumed as per standard practice :

Design speed, V = 80 kmph

Normal pavement width, W = 7.0 m

Allowable rate of change of centrifugal acceleration, (range of value 0.5 to 0.8)

$$C = \frac{80}{(75+V)} = \frac{80}{75+80} = 0.52$$

As the value of C is between 0.5 and 0.8 it is accepted for design.

Allowable rate of introduction of superelevation = 1 in 150, pavement to be rotated about the inner edge to effect better drainage in heavy rain fall area.

(a) Length of transition curve by rate of change of centrifugal acceleration :

$$L_s = \frac{v^3}{C\,R} = \frac{0.0215\,V^3}{C\,R} = \frac{0.0215 \times 80^3}{0.52 \times 500} = 42.3 \text{ m}$$

(b) Length of transition curve by the rate of introduction of superelevation :

$$e = \frac{V^2}{225\,R} = \frac{80^2}{225 \times 500} = 0.057 \; (< 0.07, \text{ O.K.})$$

Extra widening at curve (assuming two lanes and wheel base of 6 m)

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}} = \frac{2 \times 6^2}{2 \times 500} + \frac{80}{9.5\sqrt{500}} = 0.45 \text{ m}$$

Total width of pavement = B = 7.0 + 0.45 = 7.45 m

$$L_s = 7.45 \times 0.057 \times 150 = 63.7 \text{ m}$$

(c) Check for minimum value of L_s by Eq. 4.25 a,

$$L_s = \frac{2.7 V^2}{R} = \frac{2.7 \times 80^2}{500} = 34.6 \text{ m}$$

Adopt the highest of the above three values = 63.7 say, 64 m. Therefore, the design length of transition curve is 64 m.

4.4.8 Set-back Distance on Horizontal Curves

In the design of horizontal alignment, the sight distance along the inner side of the curves should be considered. Where there are sight obstruction like buildings, cut slopes, or tree on the inner side of the curves, either the obstruction should be removed or the alignment should be changed in order to provide adequate sight distance. It may some times be possible to make some adjustments in the normal highway cross section to make up small deficiencies in sight distance. If it is not possible to provide the adequate sight distance on curves on existing roads, regulatory and cautionary signs should be installed to control the traffic suitably. In case of new highways for the design speed and distance requirements, the actual condition in the alignment should be checked and necessary adjustments be made in a manner most fitting to provide adequate sight distance. Specific study is usually necessary for each site condition.

As discussed in Art. 4.3, the absolute minimum sight distance which is the safe stopping sight distance should be available at every section of the highway, from safety point. Thus it is essential that in horizontal alignment, special care should be taken to provide for the stopping sight distance; these values may be adopted as given in Table 4.5 for the design speed. Overtaking sight distance requirements are given in Table 4.7. The *clearance distance* or *set back distance* required from the centre line of a horizontal curve to an obstruction on the inner side of the curve to provide adequate sight distance depends upon the following factors :

(i) required sight distance, S

(ii) radius of horizontal curve, R

(iii) length of the curve, L_c which may be greater or lesser than S.

Refer Fig. 4.31. Let C be the obstruction to vision on the inner side of a horizontal highway curve of radius R, ABC the line of sight and arc AB be the sight distance S.

(a) $L_c > S$

Let the length of curve L_c be greater than the sight distance S. The angle subtended by the arc length S at the centre be α. On narrow roads such as single lane roads, the sight distance is measured long the centre line of the road and the angle subtended at the centre, α is equal to S/R radians.

Therefore half central angle is given by :

$$\frac{\alpha}{2} = \frac{S}{2R} \text{ radians} = \frac{180\ S}{2\pi R} \text{ degrees}$$

The distance from the obstruction to the centre is R cos α/2. Therefore the set-back distance, m required from the centre line is given by :

$$m = R - R\cos\frac{\alpha}{2} \qquad (4.27a)$$

In the case of wide roads with two or more lanes, if d is the distance between the centre line of the road and the centre line of the inside lane in metre, the sight distance is measured along the middle of the inner side lane and the set-back distance, m' is given by :

$$m' = R - (R-d)\cos\frac{\alpha'}{2} \qquad (4.27b)$$

where $$\frac{\alpha'}{2} = \frac{180\ S}{2\pi(R-d)} \text{ degrees}$$

(b) $L_c < S$

If the sight distance required is greater than the length of curve L_c, then the angle α subtended at the centre is determined with reference to the length of circular curve, L_c and the set-back distance is worked out in two parts as given below : See Fig. 4.33.

$$\frac{\alpha}{2} = \frac{180\ L_c}{2\pi(R-d)} \text{ degrees}$$

$$m' = R-(R-d)\cos\frac{\alpha'}{2} + \frac{(S-L_c)}{2}\sin\frac{\alpha'}{2} \qquad (4.28)$$

The clearance of obstruction upto the set-back distance is important when there is cut slope on the inner side of the horizontal curve. The method of calculating the set-back distance is illustrated in Examples 4.19 and 4.20.

4.4.9 Curve Resistance

The automobiles are steered by turning the front wheels, but the rear wheels do not turn. When a vehicle driven by rear wheels moves on a horizontal curve, the direction of rotation of rear and front wheels are different, as shown in Fig. 4.30 and so there is some loss in the tractive force.

A and B are the rear driving wheels which give a tractive force T in the direction PQ. The front wheels C and D are turned so as to steer the vehicle along a horizontal curve, the tangential direction of which is RS. Hence the tractive force available in this direction = T cos α which will be less than the actual tractive force, T applied. Obviously when the turning radius is sharp, the turning angle will be high and the value of T cos α would decrease. Thus the loss of tractive force due to turning of a vehicle on a horizontal curve, which is termed as *curve resistance* will be equal to (T – T cos α) or T (1 – cos α) and will depend on the turning angle α.

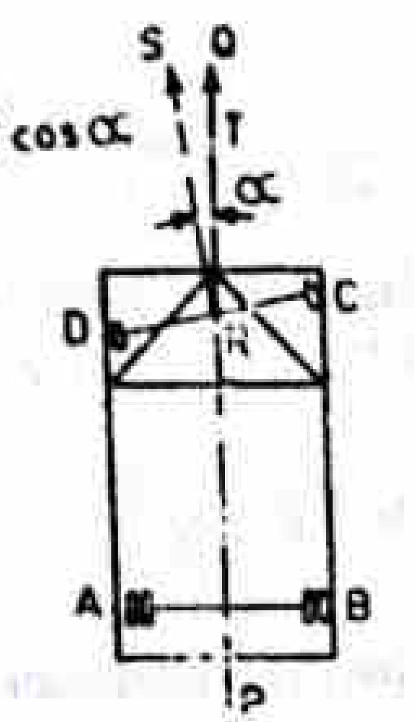


Fig. 4.30 Curve Resistance for Turning Vehicle

Turning along the horizontal curve is effected due to the lateral friction developed between the front wheels and the pavement. At sharp curves, if the speed is high there may be even sliding along the tangential direction PQ. Thus while a vehicle driven by rear wheels turns along a horizontal curve, there is increased resistance and if the same speed as on straight is to be maintained, a higher tractive effort is needed. But in the vehicles with front driving wheels, this problem does not exist. Most of the heavy commercial vehicles have rear driving wheels and hence on sharp curves the additional curve resistance should also be considered while designing the geometric features of highways. This problem of curve resistance is acute on hill roads as the curves are often sharp and in addition, the roads have steep gradients. The compensation in gradient needed for such a case has been explained later in Art. 4.5.2.

4.4.10 General Examples on Horizontal Alignment

Example 4.18

While aligning a highway in a built up area, it was necessary to provide a horizontal circular curve of radius 325 metre. Design the following geometric features :

(i) Superelevation

(ii) Extra widening of pavement

(iii) Length of transition curve

Data available are

Design speed = 65 kmph, Length of wheel base of largest truck = 6 m, Pavement width = 10.5 m

Solution

(i) Superelevation rate, e

From practical considerations of mixed traffic conditions, superelevation to fully counteract centrifugal force should be designed with 75% of design speed. Here radius R = 325 m.

$$e = \frac{V^2}{225\,R} = \frac{65^2}{225 \times 325} = 0.058$$

As this value is less than 0.07, it is safe for the design speed.

Hence provide superelevation rate = 0.058.

(ii) Extra widening of pavement, W_e

$$W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$$

n = 3 as the pavement width is 10.5 m

Wheel base = 6 m

$$W_e = \frac{3 \times 6^2}{2 \times 325} + \frac{65}{9.5\sqrt{325}}$$

$$= 0.166 + 0.380 = 0.546, \text{ say, } 0.55 \text{ m}$$

(iii) Length of transition curve, L_s

(a) By rate of change of centrifugal acceleration :

Allowable rate of change of centrifugal acceleration C is given by :

$$C = \frac{10}{75 + V} = \frac{80}{76 + 65} = 0.57 \text{ m/sec}^3$$

(As this value is between 0.5 and 0.8, accepted for design)

$$L_s = \frac{0.0215\,V^3}{C\,R} = \frac{0.0215 \times 65^3}{0.57 \times 325} = 31.9 \text{ m}$$

(b) By rate of introduction of superelevation, E : Total superelevation, E = B × e.

Total pavement width including extra widening on curve,

$$B = W + W_e = 10.5 + 0.55 = 11.05 \text{ m}$$

Superelevation rate, $e = 0.058$

$$E = 11.05 \times 0.058 = 0.64 \text{ m}$$

Assuming that superelevation is provided by rotating about the centre line, the total superelevation to be distributed along the length of transition curve = E/2. The rate of introduction of superelevation may be taken as 1 in 100, being built up area.

Length of transition curve $L_s = \frac{0.64}{2} \times 100 = 32$ m

(c) By IRC formula, the minimum length

$$L_s = \frac{2.7\,V^2}{R} = \frac{2.7 \times 65^2}{325} = 35.1 \text{ m}$$

Adopting highest of the above three values, length of transition curve L_s = 35 m.

Example 4.19

A State Highway passing through a rolling terrain has a horizontal curve of radius equal to the ruling minimum radius.

(i) Design all the geometric features of this curve, assuming suitable data.

(ii) Specify the minimum set-back distance from the centre line of the two lane highway on the inner side of the curve up to which the buildings etc. obstructing vision should not be constructed so that Intermediate sight distance is available throughout the circular curve. Assume the length of circular curve greater than the sight distance.

Solution

The various geometric elements to be designed are

(a) Ruling minimum radius

(b) Superelevation

(c) Extra widening

(d) Length of transition curve

(e) SSD, ISD and set-back distance

(a) Ruling minimum radius of curve for ruling design speed of 80 kmph :

$$R_{ruling} = \frac{V^2}{127(e+f)} = \frac{80^2}{127(0.07+0.15)}$$

$$= 229 \text{ m, say } 230 \text{ m}$$

(b) Design value of superelevation :

$$e = \frac{V^2}{225R} = \frac{80^2}{225 \times 230} = 0.124$$

As the value is higher than the maximum superelevation of 0.07, limit the value of e to 0.07. The curve should be safe for the full speed of 80 kmph as the ruling minimum radius has been adopted. However check the transverse skid resistance developed :

$$f = \frac{V^2}{127R} - e = \frac{80^2}{127 \times 230} - 0.07 = 0.149$$

(Less than 0.15 and hence safe)

(c) Assume two lane pavement, i.e., n = 2 and l = 6 m. Extra widening of pavement,

$$W_e = \frac{nl^2}{2R} + \frac{V}{0.5\sqrt{R}}$$

$$= \frac{2 \times 6^2}{2 \times 230} + \frac{80}{9.5\sqrt{230}} = 0.157 + 0.555 = 0.712 \text{ m}$$

Provide an extra width of 0.71 m and a total width of pavement B = 7.71 m.

(d) Length of transition curve is designed by calculating the values based on (i) rate of change of centrifugal acceleration C (ii) rate of introduction of the amount of superelevation E and (iii) minimum length formula; the highest of three values is adopted at the design length L_s.

(i)
$$C = \frac{80}{75+V} = \frac{80}{75+80} = 0.52$$

(as this is within the range 0.5 to 0.8, the value is acceptable for design).

$$L_s = \frac{0.0215\,V^3}{CR} = \frac{0.0215 \times 80^3}{0.52 \times 230} = 92 \text{ m}$$

(ii) Total amount of superelevation E i.e., the raising of the outer edge of the pavement with respect to inner edge = B × e = 7.71 × 0.07 = 0.54 m. As the terrain is rolling, assume the pavement to be rotated about the centre at a rate of 1 in 150.

$$L_s = \frac{E}{2} \times N = \frac{0.54 \times 150}{2} = 40.5 \text{ m}$$

(iii) Minimum value of L_s as per IRC is given by :

$$L_s = \frac{2.7V^2}{R} = \frac{2.7 \times 80^2}{230} = 75.1 \text{ m}$$

Adopting the highest of the three values, design length of transition curve = 92 m.

(e) Intermediate Sight Distance = 2 SSD

$$= 2\left[0.278\,Vt + \frac{V^2}{254f}\right]$$

$$= 2\left[0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}\right] = 2 \times 127.6 = 255 \text{ m}$$

(f) Refer Fig. 4.31. The length of circular curve is assumed greater than the desired sight distance SD. The minimum clearance or set-back distance needed m = CD and half the central angle-α'/2 = angle AOD.

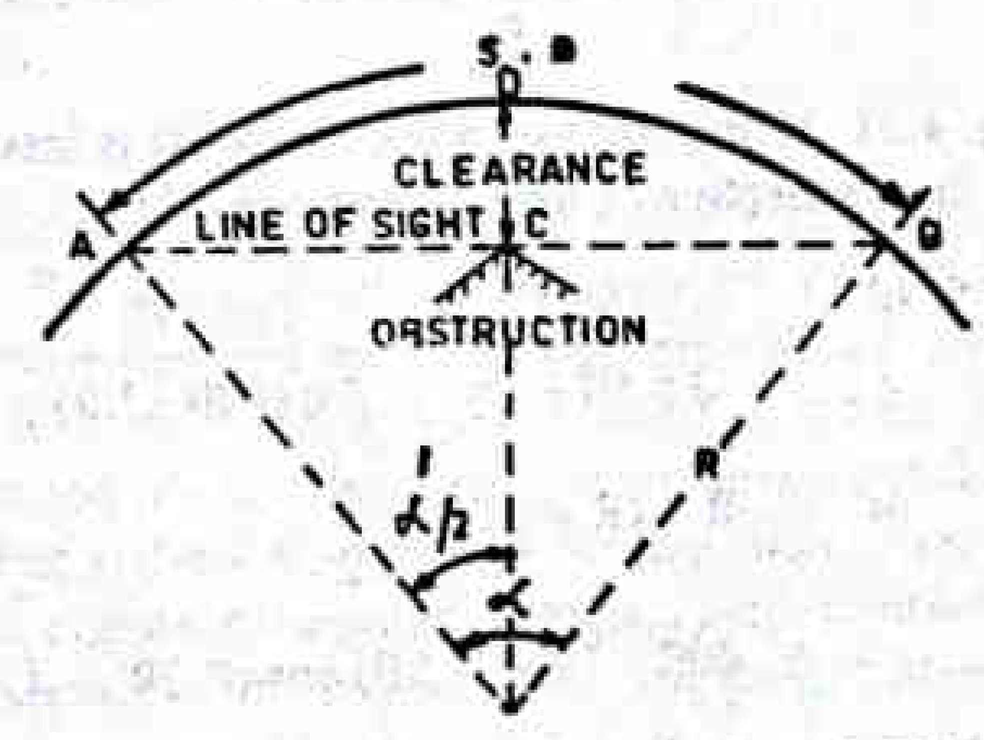


Fig. 4.31 Clearance on curve or Set-back Distance for Desired Sight Distance (Example 4.19)

The distance d between the centre line of the pavement and the centre line of the inside lane may be taken as one-fourth the width of pavement at the curve (being a two-lane pavement) = 7.71/4 = 1.93 m.

$$\frac{\alpha'}{2} = \frac{180\,S}{2\pi\,(R-d)} = \frac{180\times 255}{2\pi\,(230-1.93)} = 32^\circ$$

$$\text{Set-back distance } m' = R-(R-d)\cos\frac{\alpha'}{2}$$

$$= 230-(230-1.93)\cos 32^\circ = 36.6 \text{ m}$$

Therefore the minimum set-back distance or clearance required to provide a clear vision for an ISD of 255 m is 36.6 m.

Example 4.20

There is a horizontal highway curve of radius 400 m and length 200 m on this highway. Computer the set-back distances required from the centre line on the inner side of the curve so as to provide for

(a) stopping sight distance of 90 m

(b) safe overtaking sight distance of 300 m.

The distance between the centre lines of the road and the inner lane is 1.9 m.

Solution

(a) Refer Fig. 4.32. The stopping sight distance (SSD) of 90 m is less than the circular curve length of 200 m

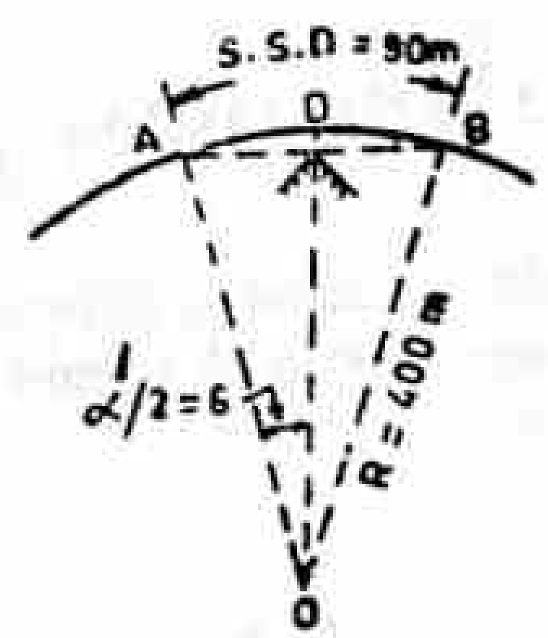


Fig. 4.32 Minimum Set-back when SD is less than Length of Curve (Example 4.20a)

$$\frac{\alpha'}{2} = \frac{180\,S}{2\pi\,(R-D)} = \frac{180\times 90}{2\pi\,(400-1.9)} = 6^\circ\,29'$$

$$m' = R-(R-d)\cos\frac{\alpha'}{2}$$

$$= 400-(400-1.9)\cos 6^\circ\,29' = 4.4 \text{ m}$$

Required clearance from the centre line to provide SSD of 90 m is 4.4 m.

(b) Refer Fig. 4.33. The overtaking sight distance of 300 m is greater than circular curve length which is 200 m. Therefore the required set-back distance is CF = (CG + GF) and is given by Eq. 4.28.

$$S = 300 \text{ m},\ L_c = 200 \text{ m},\ R = 400 \text{ m},\ d = 1.9 \text{ m}$$

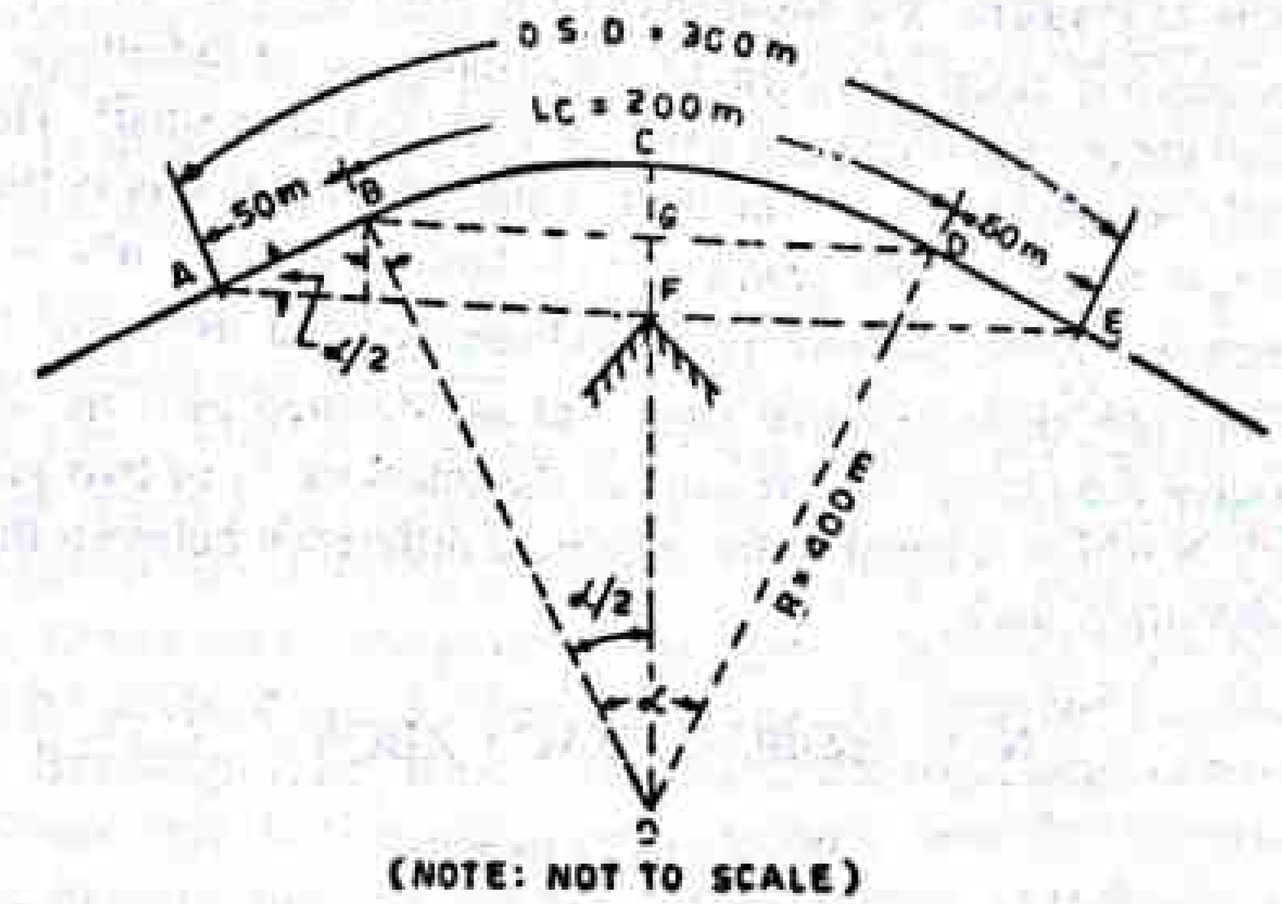


Fig. 4.33 Minimum Set-back Distance when SD is Greater than Length of Curve (Example 4.20b)

$$\frac{\alpha'}{2} = \frac{180\,L_c}{2\pi\,(R-d)} = \frac{180\times 200}{2\pi\,(400-1.9)} = 14.39^\circ$$

$$\text{Set-back distance } m' = CF = CG + GF$$

$$= R-(R-d)\cos\frac{\alpha'}{2} + \frac{(S-L_c)}{2}\sin\frac{\alpha'}{2}$$

$$= 400-(400-1.9)\cos 14.39^\circ + \frac{(300-200)}{2}\sin 14.39^\circ$$

$$= 14.4 + 12.4 = 26.8$$

Minimum set-back distance required from the centre line of the roads on the inner side of the pavement to provide an OSD of 300 m = 27 m.

4.5 DESIGN OF VERTICAL ALIGNMNET

4.5.1 General

While aligning a highway it is the general practice to follow the general topography or profile of the land. But the natural ground may be level only at some places and otherwise the ground may have slopes of varying magnitudes. Hence the vertical profile of a road would have level stretches as well as slopes or grades. In order to have smooth vehicle movements on the roads, the changes in the grade should be smoothened out by the vertical curves. The vertical alignment is the elevation or profile of the centre line of the road. The vertical alignment consists of grades and vertical curves, and it influences the vehicle speed, acceleration, deceleration, stopping distance, sight distance and comfort in vehicle movements at high speeds.

4.5.2 Gradient

Gradient is the rate of rise or fall along the length of the road with respect to the horizontal. It is expressed as a ratio of 1 in x (1 vertical unit to x horizontal units). Some times the gradient is also expressed as a percentage, n i e, n in 100.

When the angle of gradient, α is small (Refer Fig. 4.34a) the gradient which is 1 in α or tan α is approximately equal to the circular measure or α in radians (α^c). All angles within the practical range of gradients on roads may be treated as small. Hence gradients which are generally represented as 'n' percent, would mean that this is the value of the tangent of the angle made by the gradient with horizontal, i.e., n% = tan α. The ascending gradients are given positive signs and are denoted as + n_1, + n_2 etc., and descending gradients are given negative signs and are denoted as − n_3, − n_4 etc. The angle which measures the change of direction at the intersection of two grades is called the *deviation angle* N which is equal to the algebraic difference between the two grades. In Fig. 4.34b the deviation angle,

$$N = \angle DBC = \angle BAC + \angle BCA$$

$$= +n_1 - (-n_2) = n_1 + n_2$$

where n_1 is ascending gradient of AB and − n_2 the descending gradient of BC.

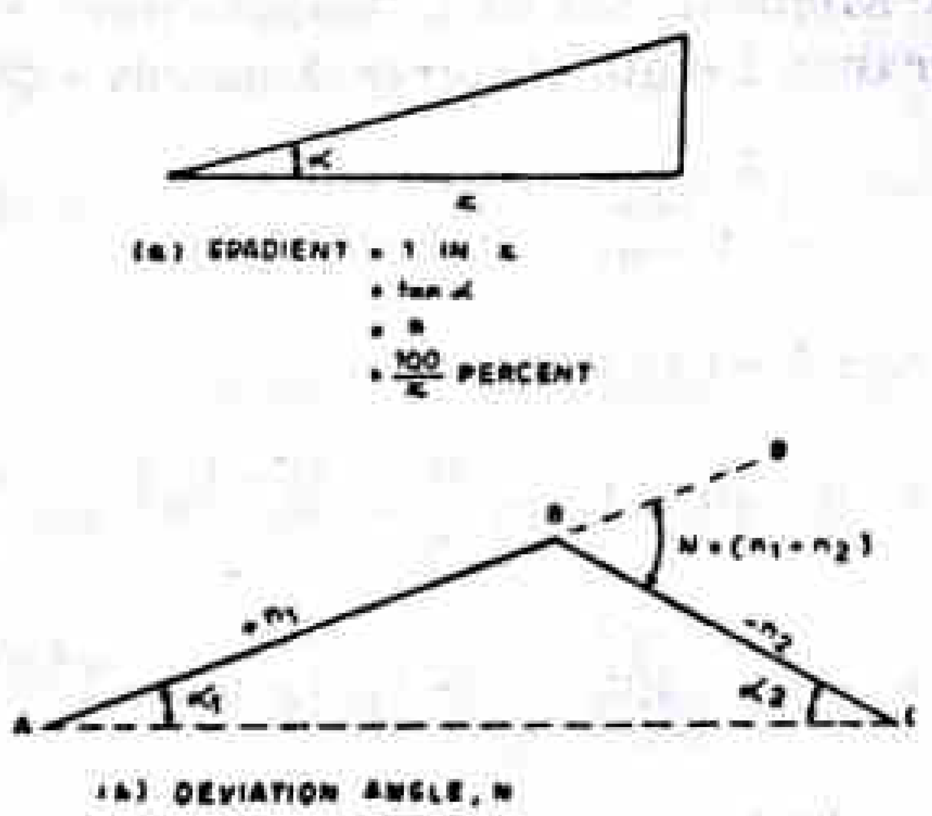


Fig. 4.34 Measure of Gradients

While aligning a highway, the gradient is decided for designing the vertical curve. Very steep gradients are avoided as it is not difficult to climb the grade, but also the vehicle operation cost is increased. The engineer has to consider all aspects such as construction costs, practical problems in construction at the site and the vehicle operation cost in such alternative proposals before finalising the gradients.

Gradients are divided into the following categories :

Ruling gradient

Limiting gradient

Exceptional gradient

Minimum gradient

The terms are explained below :

Ruling gradient is the maximum gradient within which the designer attempts to design the vertical profile of a road. Gradients up to the ruling gradient are adopted as a normal course in design of vertical alignment and accordingly the quantities of cut and fill are balanced. Hence ruling gradient is also known as design gradient. However flatter gradients may be preferred where ever practicable.

The selection of ruling gradient for the purpose of design is a complex job as several factors such as type of terrain, the length of the grade, the speed, pulling power of vehicles and presence of horizontal curves are considered. In flat terrain or plain country it may be possible to adopt a flat gradient : but in hill roads it may not be economical or some time not even possible to adopt the same gradient because of large difference in levels to be covered in short length of road.

A vehicle which travels with a certain speed on a level ground, with the same tractive effort put in, would lose speed at grades; the speed would steadily decrease with increase in length of grade. With the maximum pulling power, the vehicle would be able to sustain the same speed even on long sections only up to a certain gradient. This is when the maximum power developed by the engine is equal to the power required to overcome the resistances to motion on the grade at this speed. Therefore this gradient, is the one which should be adopted at a ruling gradient by the designer for this vehicle and the design speed. But the problem is not so simple as different vehicles have different values of hauling power and varying tractive resistances and the commercial vehicles in particular have to carry different amount of load. Further in India due consideration is to be given to the pulling power of animal drawn vehicle, especially the bullock carts.

Thus it is not possible to lay down precise standards of ruling gradient applicable for the mixed traffic and for the country as a whole.

The IRC has recommended ruling gradient values of 1 in 30 on plain and rolling terrain, 1 in 20 on mountainous terrain and 1 in 16.7 on steep terrain.

Where topography of a place compels adopting steeper gradients than ruling gradients. *limiting gradients* are used in view of enormous increase in cost in constructing roads with gentler gradients. However the length of continuous grade line steeper than ruling value should be limited. On rolling terrain and on hill roads, it may be frequently necessary to exceed ruling gradient and adopt limiting gradient; but care should be taken to separate such stretches of steep gradients by providing either a level road or a road with easier grade.

In some extra ordinary situations it may be unavoidable to provide still steeper gradients atleast for short stretches and in such cases the steeper gradient upto *exceptional gradients* may be provided. However the exceptional gradient should be strictly limited only for short stretches not exceeding about 100 m at a stretch.

The maximum length of ascending gradient which a loaded truck can operate without undue reduction in speed is called *critical length of grade* for a design. A reduction in speed of about 25 kmph may be considered reasonable limit. The critical length for design depends on several factors such as size, power, load and grade ability data of the truck, its initial speed at the beginning of the ascending grade and the desirable limit of the minimum speed at the end of the grade so as to avoid unreasonable interference with the movement of other vehicles. The critical length of ascending gradients should therefore be limited to lower values at steeper gradients.

The maximum values of gradients recommended by the IRC at different terrains are given in Table 4.13.

The road can be level, with little or no gradient. In such cases there will be problems of drainage. Though the surface water can be drained off to the side drains by providing proper camber on the pavement surface and cross slope on shoulders, a certain longitudinal slope is essential, to drain the water along the side drains, depending on the surface of the drains.

Table 4.13 Gradients for Roads in Different Terrains

Terrain	Ruling gradient	Limiting gradient	Experimental gradient
Plain or rolling	3.3 per cent (1 in 30)	5 per cent (1 in 20)	6.7 per cent (1 in 15)
Mountainous terrain, and steep terrain having elevation more than 3,000 m above the mean sea level	5 per cent (1 in 20)	6 per cent (1 in 16.7)	7 per cent (1 in 14.3)
Steep terrain upto 3,000 m height above mean sea level	6 per cent (1 in 16.7)	7 per cent (1 in 14.3)	8 per cent (1 in 12.5)

Suppose the road is with zero gradient passing through level land and open side drains are provided with a gradient of say 1 in 300. It may then be necessary to deepen the downstream end of the drain by about 3.3 m for one kilometer length of road. This course is not possible from practical considerations. Hence it is desirable to have a certain *minimum gradient* on roads from drainage point of view, provided topography favours this. The minimum gradient would depend on rain fall run off, type of soil, topography and other site conditions.

A minimum gradient of about 1 in 500 may be sufficient to drain water in concrete drains or gutter; but on inferior surfaces of drains a slope of 1 in 200 or 0.5 percent may be needed where as on kutcha open drains (soil drains) steeper slopes upto 1 in 100 or 1.0 percent may be needed.

Compensation in Gradient on Horizontal Curves

At horizontal curves, due to the turning angle α of vehicles, the curve resistance developed is equal to $T(1 - \cos\alpha)$, as explained in Art. 4.4.9. When there is a horizontal curve in addition to the gradient, there will be increased resistance to fraction due to both gradient and curve. It is necessary that in such cases the total resistance due to grade and curve should not normally exceed the resistance due to the maximum value of the gradient specified. For design purpose, this maximum value may be taken as the ruling gradient and in some special cases as limiting gradient for the terrain. When sharp horizontal curve is to be introduced on a road which has already the maximum permissible gradient, then the gradient should be decreased to compensate for the loss of tractive effort due to the curve.

This reduction in gradient at the horizontal curve is called grade compensation, which is intended to off-set the extra tractive effort involved at the curve. This, is calculated from the relation :

Grade compensation, percent = $\dfrac{30+R}{R}$, subject to a maximum value of 75/R, where R is the radius of the circular curve in metre.

According to the IRC the grade compensation is not necessary for gradients flatter than 4.0 percent and therefore when applying grade compensation correction, the gradients need not be eased beyond 4 percent.

Example 4.21

While aligning a hill road with a ruling gradient of 6 percent, a horizontal curve of

Solution

Ruling gradient = 6.0%

Grade compensation = $\dfrac{30+R}{R} = \dfrac{30+60}{60} = 1.5\%$

Maximum limit of grade compensation = 75/R = 75/60 = 1.25%

Therefore, compensated gradient = 6.0 − 1.25 = 4.75%

4.5.3 Vertical Curves

Due to changes in grade in the vertical alignment of highway, it is necessary to introduce vertical curve at the intersections of different grades to smoothen out the vertical profile and thus ease off the changes in gradients for the fast moving vehicles.

The vertical curves used in highway may be classified into two categories :

(a) *Summit* curves or *crest* curves with convexity upwards.

(b) *Valley* or *sag* curves with concavity upwards.

Summit curves

Summit curves with convexity upwards are formed in any one of the case illustrated in Fig. 4.35. The deviation angle between the two interacting gradients is equal to the algebraic difference between them. Of all the cases, the deviation angle will be maximum when an ascending gradient meets with a descending gradient i.e., $N = n_1 - (-n_2) = (n_1 + n_2)$.

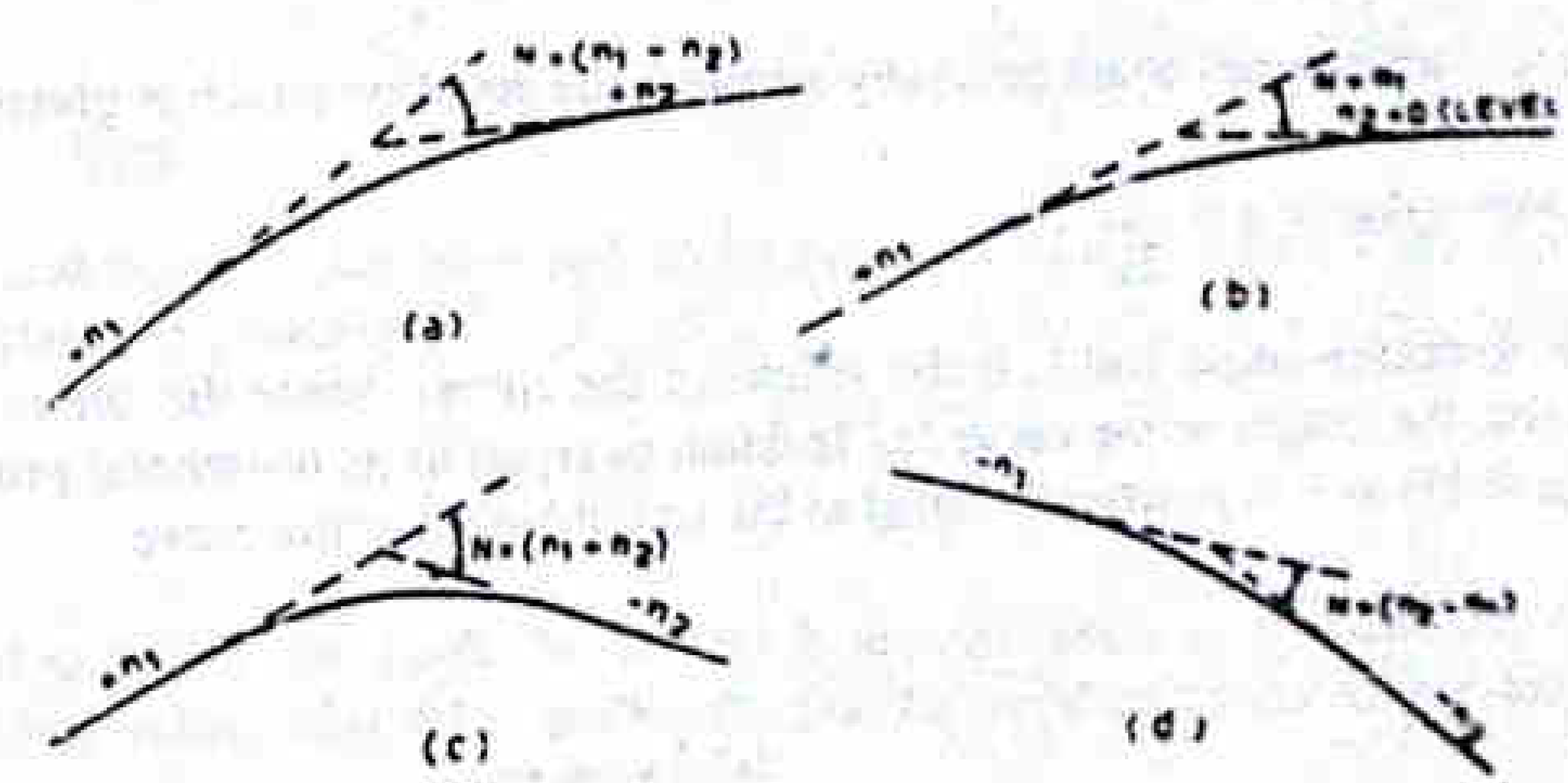


Fig. 4.35 Summit Curves

When a fast moving vehicle travels along a summit curve, the centrifugal force will act upwards, against gravity and hence a part of the pressure on the tyres and spring of the vehicle suspensions is relieved. So there is no problem of discomfort to passengers on summit curves, particularly because the deviation angles on roads are quite small and if the summit curve is designed to have adequate sight distance, the length of the curve would be long enough to ease the shock due to change in gradients.

The only problem in designing summit curves is to provide adequate sight distances. The stopping sight distance or the absolute minimum sight distance should invariably be provided at all sections of the road system and so also on summit curves. As far as possible safe overtaking sight distance or at least intermediate sight distance, equal to twice the stopping sight distance should also be available on these curves for important highways, except when restrictions to overtaking have been strictly imposed at the sections concerned.

Types of summit curves

As the design of summit curves (except low summit or *humps* which do not interfere with sight distance) are governed only by considerations of sight distance, transition curves are not necessary. Circular summit curve is ideal as the sight distance available throughout the length of circular curve is constant. From this view point, transition curve may be said to be even undesirable to be used on summit curves. This is because the radius of curvature and hence the sight distance would vary from point to point along the length of curve. The deviation angles in vertical curves of highways are very small and so between the same tangent points, a simple parabola is nearly congruent with a circular arc; also a parabola is very easy for arithmetical manipulation for computing ordinates. The use of simple parabola as summit curve is found to give good riding comfort too. Because of these reasons in actual practice a simple parabolic curve is used as summit curve instead of the circular arc.

There may be situations where the problem of sight distance does not arise as in the case when the road goes over a *hump*. This may be due to the presence of a culvert, the top of which is above the general level of the road by less than about a metre thus causing a sharp but relatively small summit or hump on the road profile. In such a case the problem is of discomfort to the passengers and so the most suitable curve would be transition curves on either side of this hump. For proper design of humps, the vertical profile should consist of two transition curves on either side of the hump with a level strip in between.

Length of summit curve

Parabolic summit curves are generally adopted, the equation which is given by :

$y = a x^2$, with value of $a = \dfrac{N}{2L}$

Here N is deviation angle and L is the length of the curve. Since the summit curve are long and flat, the length of the curve 'L' is taken as equal to its horizontal projection AH (Refer Fig. 4.36) as it is practically equal to the actual length of the curve.

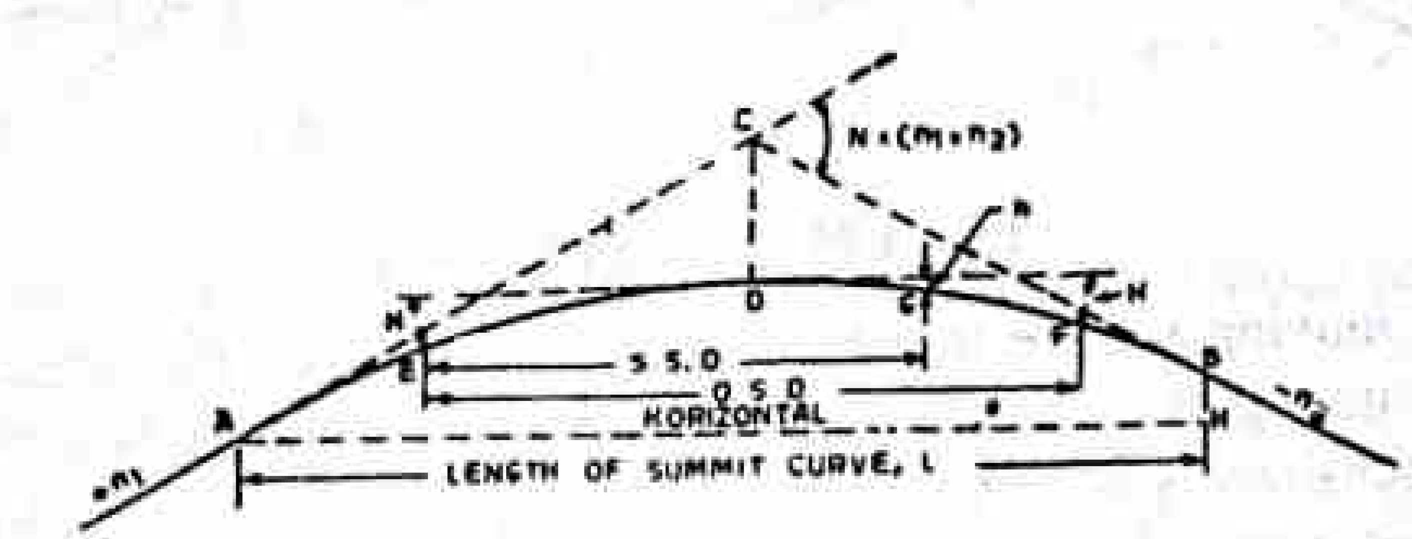


Fig. 4.36 Length of Summit Curve

While designing the length of the parabolic summit curves, it is necessary to consider the stopping sight distance (SSD) and overtaking sight distance (OSD) separately. As mentioned earlier, it is essential to provide sight distance at least equal to the stopping sight distance at all points on a highway so as to avoid accidents due to inadequate sight distance.

Length of summit curve for stopping sight distance (SSD)

Two cases are to be considered in deciding the length

(i) When the length of the curve is greater than the sight distance (L > SSD)

(ii) When the length of the curve is less than the sight distance (L < SSD)

(i) When L > SSD

The general equation for length L of the parabolic curve is given by :

$$L = \frac{N S^2}{(\sqrt{2H} + \sqrt{2h})^2} \qquad (4.29)$$

Here
- L = length of summit curve, m
- S = stopping sight distance, (SSD), m
- N = deviation angle, equal to algebraic difference in grades, radians or tangent of the deviation angle.
- H = height of eye level of driver above roadway surface, m
- h = height of subject above the pavement surface, m

The value of H, the height of driver's eye above roadway surface is taken as 1.2 m in India as discussed in Art. 4.3.2. The height of object 'h' above the pavement surface for the purpose of safe stopping distance is taken as 0.15 m as per the IRC standard. Substituting these values in equation 4.29 the length of summit curve is obtained as :

i.e. $$L = \frac{N S^2}{4.4} \qquad (4.30)$$

(ii) When L < SSD

The general equation for the length of the parabolic summit curve, when it is less than the sight distance is given by :

$$L = 2S - \frac{(\sqrt{2H} + \sqrt{2h})^2}{N} \qquad (4.31)$$

Here the description for L, S, N, H and h are the same as in equation 4.29. By substituting the values of H = 1.2 and h = 0.15 m the length of the curve is obtained as :

$$L = 2S - \frac{4.4}{N} \qquad (4.32)$$

Thus to design the length of vertical summit curve providing, for stopping or absolute minimum sight distance, first the safe stopping sight distance is found either by calculation as given in Art. 4.3.2, and equation 4.2 or from the recommended values given in Table 4.5. By using any of the two appropriate formulae by trial (4.30 and 4.32) the required length of summit curve is then calculated. The minimum radius of the parabolic summit curve may be calculated from the relation R = L/N.

Length of summit curve for safe overtaking sight distance (OSD) or Intermediate Sight Distance (ISD)

Two cases to be considered in deciding the length are :

(i) When the length of curve is greater than the overtaking or Intermediate sight distance (L > S) and

(ii) When the length of the curve is less than the overtaking or Intermediate sight distance (L < S)

(i) When L > S

The same general Eq. 4.29 is applicable in this case also. But in this case, the values of H and 'h' both are taken equal to 1.2 m. Substituting h = H in the Eq. 4.29 and simplifying.

$$L = \frac{NS^2}{8H}$$

As discussed in Art. 4.3.3 the height of the eye level of the driver as well as the height of the approaching object are taken as 1.2 m. Substituting the value of H, the height of eye level of driver above the pavement surface equal to 1.2 m.

$$L = \frac{NS^2}{9.6} \qquad (4.33)$$

Here, L = the length of parabolic summit curve,

N = deviation angle, radians or tangent of the deviation angle,

S = overtaking or intermediate sight distance, (OSD/ISD)

(ii) When L < S

The same general equation 4.31 may be used. By substituting H = h and simplifying when L is less than OSD/ISD :

$$L = 2S - \frac{8H}{N}$$

Here again substituting the value of H as 1.2 m, the equation reduces to :

$$L = 2S - \frac{9.6}{N} \qquad (4.34)$$

Here L, S and N are as in Eq. 4.33.

Thus to design the length of vertical summit curve providing for safe or cautious overtaking, the value of the overtaking or intermediate sight distance is calculated from the available data as given in Art. 4.3.3 and Eq. 4.6 or from the recommended value given in Table 4.7 or twice the SSD value as given in Table 4.5, for finding the value of ISD. By using any of the two appropriate formula 4.33 and 4.34, the required length of summit curve is then calculated. The minimum radius of the curve R is calculated from the relation R = L/N as before.

When the deviation angle is small, the length of summit curve generally works out less than the sight distance. In very small deviation angles, the length required some times works out as a negative value indicating that there is no problem of sight restriction at the summit curve.

The minimum lengths of vertical curve for different speeds and for the maximum grade change values (in percent) which do not require vertical curve, as per the ... Table 4.14.

Table 4.14 Minimum Length of Vertical Curves

Design speed (kmph)	Maximum grade change (percent) not requiring a vertical curve	Minimum length of vertical curve, m (for higher grade change values)
35	1.5	
40	1.2	15
50	1.0	20
65	0.8	30
80	0.6	40
100	0.5	50
		60

The highest point on the summit curve is a distance L n_1/N from the tangent point on the first grade n_1. This is obtained by assuming the summit curve to be a simple parabola of equation : $y = a x^2$ and by differentiating the height y with respect to distance x and equating to zero for the highest point. The summit vertical curve may be plotted by finding the ordinate value Y_1, Y_2, Y_3 ... for different lengths X_1, X_2, X_3 .... from the tangent point, by taking measurements along the tangent lengths.

Valley curves

Valley curves or sag curves are formed in any one of the cases illustrated in Fig. 4.37. In all the cases the maximum possible deviation angle is obtained when a descending gradient meets with an ascending gradient.

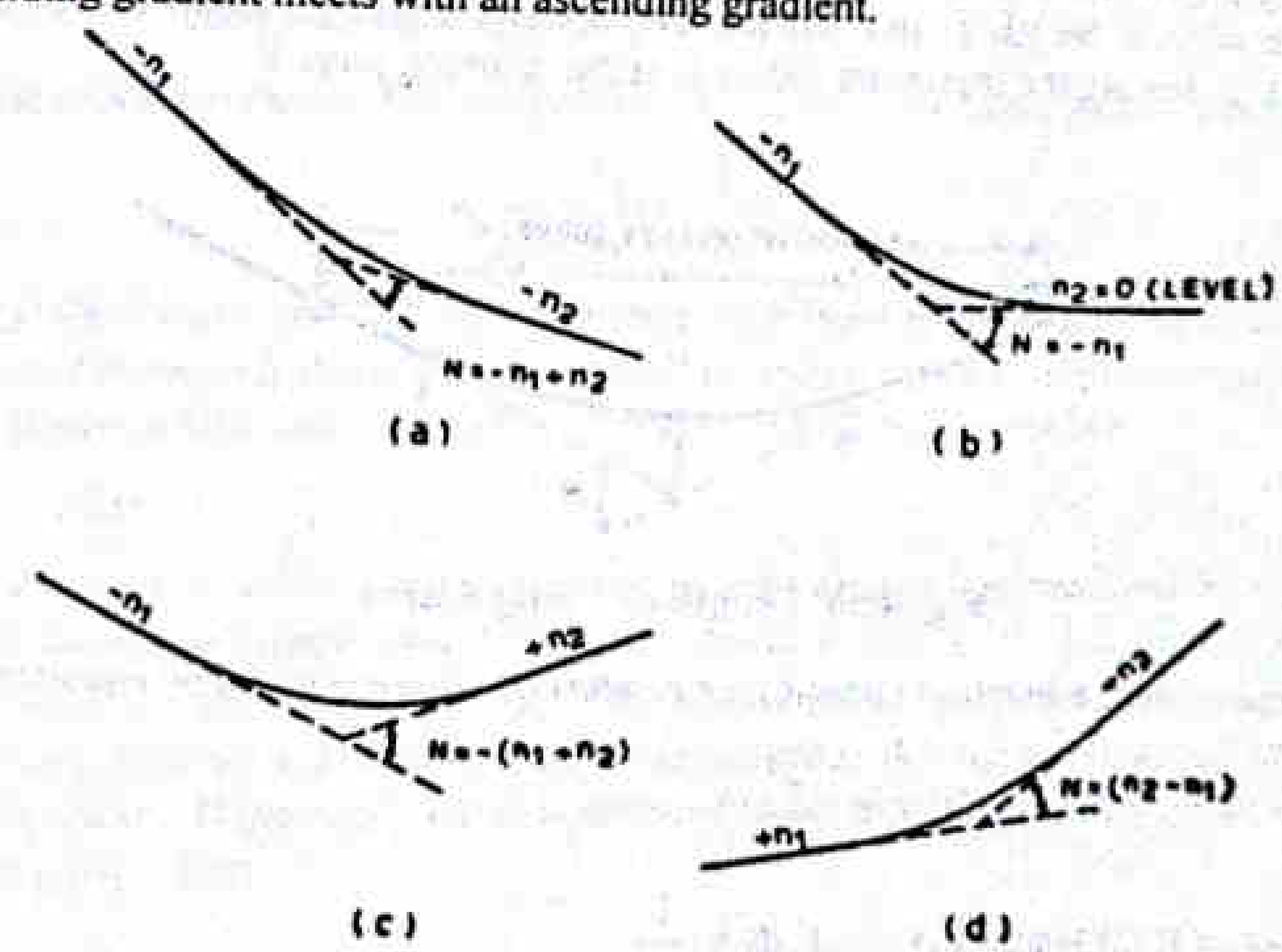


Fig. 4.37 Valley Curves

There is no problem of restriction to sight distance in valley curves during day light. However, during night driving under head lights of vehicles, the sight distance available at valley curve is decreased. The most important factors considered in valley curve design are, (i) impact-free movement of vehicles at design speed or the comfort to the passengers and (ii) availability of stopping sight distance under head lights of vehicles for night driving. The lowest point in the valley curve may be located from considerations of cross drainage.

At the valley curve, the centrifugal force acts downwards adding to the pressure on the springs and the suspensions of the vehicle in addition to that due to weight of the vehicle. Hence the allowable rate of change of centrifugal acceleration should govern the design of the valley curves. Obviously the best shape of valley curve is a transition curve for gradually introducing and increasing the centrifugal force acting downwards. Cubic parabola is generally preferred in vertical valley curves. As the deviation angles are small, the path traversed by the three types of transition curves, viz. spiral, lemniscate and cubic parabola are almost the same as explained in Art. 4.4.7.

During night driving the visibility ahead is dependent on the head light of the vehicles, when the road lighting is not adequate or has not been provided. There is restriction of the sight distance at valley curves as the head light gets intercepted though the beam of light may be slightly inclined upwards. Therefore the *head light sight distance* available at valley curves should be atleast equal to the stopping sight distance. However, there is no problem of overtaking sight distance at valley curves during night as other vehicles with head lights can be seen from a considerable distance.

Length of valley curve

The length of valley transition curve is designed based on the two criteria : (i) the allowable rate of change of centrifugal acceleration of 0.06 m/sec^3 and (ii) the head light sight distance, and the higher of the two values is adopted. Usually the second criterion of head light sight distance is higher and therefore governs the design.

The valley curve is made fully transitional by providing two similar transition curves of equal length (without a circular curve in between). Refer Fig. 4.38, where ABC is the valley curve of total length L and AB and BC are two equal transition curves each of length $L_s = L/2$, having the minimum radius R at the common point B.

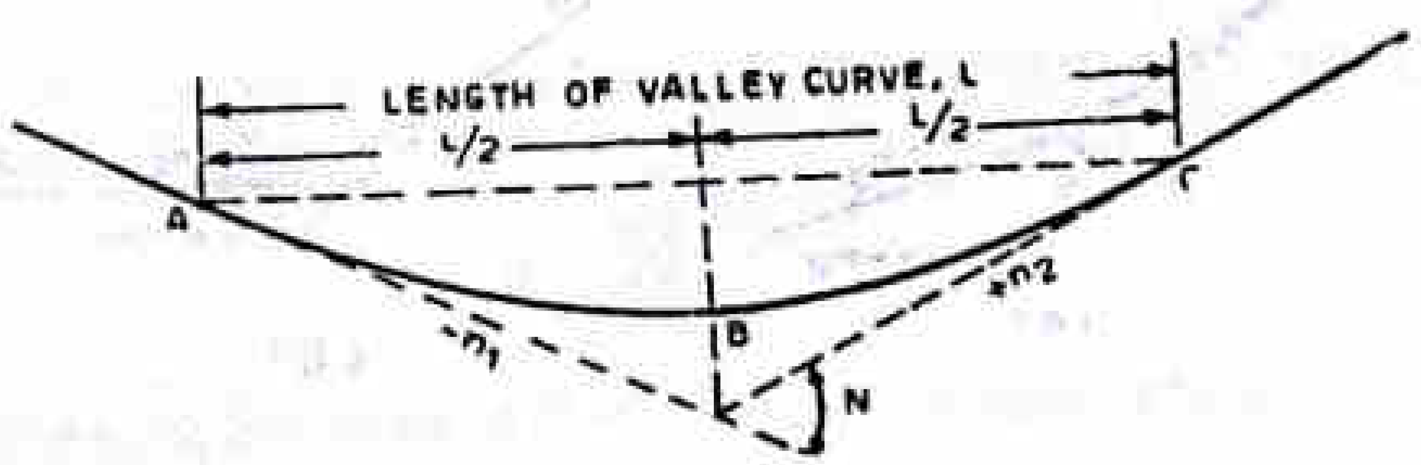


Fig. 4.38 Length of Valley Curve

(1) *The length of transition curve L_s for comfort condition* is given by equation 4.22.

$$L_s = \frac{v^3}{CR}$$

$$\text{Value of R (at length } L_s) = L_s/N = \frac{L}{2N}$$

$$L_s = \frac{v^3}{C.L_s} \times N \text{ or } L_s^2 = \frac{Nv^3}{C}$$

$$L_s = \left[\frac{Nv^3}{C}\right]^{\frac{1}{2}}$$

$$L = 2\,L_s = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} \qquad (4.35)$$

where N is the deviation angle, v is speed in m/sec and C is the allowable rate of change of centrifugal acceleration which may be taken as 0.6 m/sec^3.

$$V \text{ kmph} = \frac{V}{3.6} \text{ m/sec}$$

$$L_s^2 = \frac{NV^3}{0.6\times3.6^3}$$

$$L_s = 0.19\,(NV^3)^{1/2}$$

$$L = 2\,L_s = 0.38\,(NV^3)^{1/2} \qquad (4.36)$$

Hence the total length of valley curve is given by equation :

$$L = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} = 0.38\,(NV^3)^{1/2}$$

where L = total length of valley curve,

N = deviation angle in radius or tangent of the deviation angle or the algebraic difference in grades.

V = design speed, kmph

The minimum radius (R metre) of the valley curve for cubic parabola is given by :

$$R = \frac{L_s}{N} = \frac{L}{2N} \qquad (4.37)$$

(2) *The length of valley curve for head light sight distance* may be determined for the two conditions : (i) when the total length of valley curve L is greater than the stopping sight distance SSD and (ii) when L is less than SSD, as given below.

(i) *L > SSD*

The length of valley curve L is assumed to be greater than the head light sight distance which should be atleast equal to SSD, as shown in Fig. 4.39. Let the height of the head light be h_1 and the focused portion of the beam of light be inclined at an angle α upwards. The sight distance available will be minimum when the vehicle is at the lowest point on the sag curve. If the valley curve is assumed to be of parabolic shape, with equation $y = a x^2$, where $a = N/2L$.

$$h_1 + S\tan\alpha = a\,S^2 = \frac{N\,S^2}{2L}$$

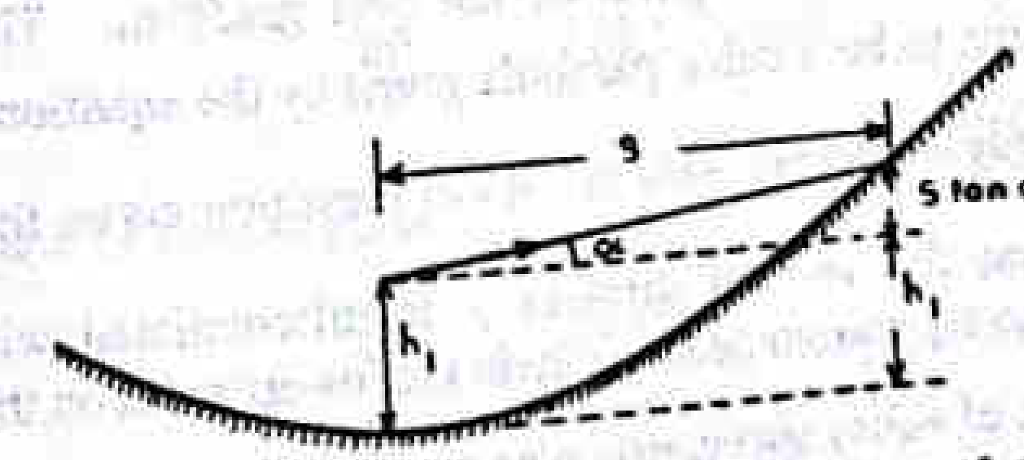


Fig. 4.39 Head Light Sight Distance when L > S

$$L = \frac{N S^2}{(2h_1 + 2 S \tan\alpha)}$$

If the average height of the head light is taken as $h_1 = 0.75$ m and the beam angle $\alpha = 1°$, by substituting these in the above equation,

$$L = \frac{N S^2}{(1.5 + 0.035 S)} \quad (4.38)$$

where, L = total length of valley curve, m (L > S)

S = SSD, m

N = deviation angle = $(n_1 + n_2)$, with slopes $-n_1$ and $+n_2$

(ii) L < SSD

Refer Fig. 4.40. Let the vehicle be at the start of the valley curve or at the tangent point TP, for minimum sight distance. Therefore,

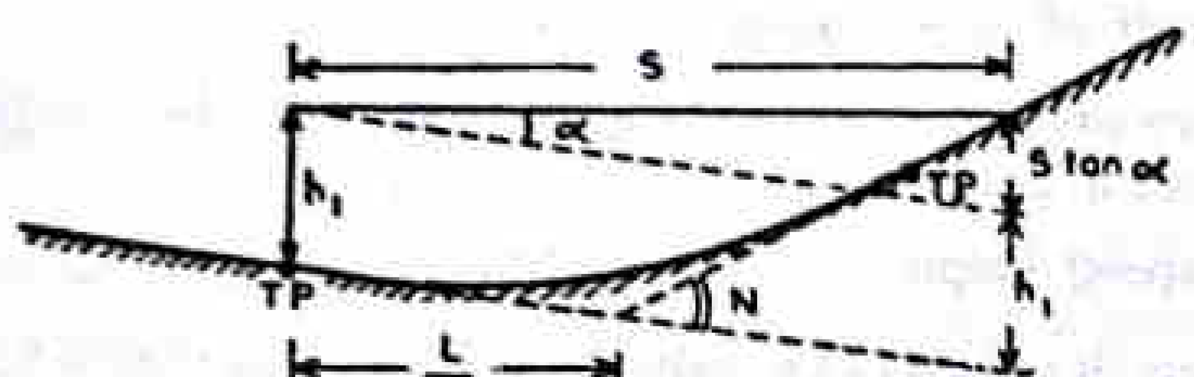


Fig. 4.40 Head Light Sight Distance when L < S

$$h_1 + S \tan\alpha = \left(S - \frac{L}{2}\right) N$$

$$L = 2S - \frac{(2h_1 + 2S\tan\alpha)}{N}$$

Substituting $h_1 = 0.75$ m and $\alpha = 1°$, when L < S

$$L = 2S - \frac{(1.5 + 0.035 S)}{N} \quad (4.39)$$

The lowest point on the valley curve is to be located for providing the cross drainage facility. The lowest point on the valley curve will be on the bisector of the angle between the grades, if the gradients on either side are equal. When the gradients are not equal, the lowest point lies on the side of flatter grade, and this point is at a distance $X_0 = L\sqrt{n_1/2N}$ from the tangent point of the first grade n_1. This is obtained by assuming the valley curve to be a cubic parabola given by the equation; $y = bx^3$, where b $= \frac{2N}{3L^2}$. This practically coincides with the spiral transition curve for small deflection angles of valley curves; the vertical distance y is differentiated with respect to the horizontal distance x and equated to zero to obtain the lowest point on the curve.

The minimum length of valley curve, may also be obtained from Table 4.14.

Example 4.22

A vertical summit curve is formed at the intersection of two gradients, + 3.0 and − 5.0 percent. Design the length of summit curve to provide a stopping sight distance for a design speed of 80 kmph. Assume other data

Solution

(i) $$SSD = 0.278\, V t + \frac{V^2}{254 f}$$

As there is ascending gradient on one side of the summit and descending gradient on the other side, the effect of gradients on the SSD is assumed to get compensated and hence ignored in the calculations.

Assuming t = 2.5 sec and f = 0.35 for V = 80 kmph,

$$SSD = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35}$$

$$= 55.6 + 72.0 = 127.6, \text{ say } 128 \text{ m}$$

(ii) Deviation angle $N = 0.03 - (-0.05) = 0.08$

Assuming L > SSD (Eq. 4.30),

$$L = \frac{N S^2}{4.4} = \frac{0.08 \times 128^2}{4.4}$$

$$= 297.9 \text{ m } (> 128 \text{ m})$$

Therefore length of summit curve = 298 m

This value is higher than the minimum specified length of 50 m at 80 kmph speed as per Table 4.14.

Example 4.23

An ascending gradient of 1 in 100 meets a descending gradient of 1 in 120. A summit curve is to be designed for a speed of 80 kmph so as to have an overtaking sight distance of 470 m.

Solution

$$n_1 = +\frac{1}{100}, \; n_2 = -\frac{1}{120}$$

$$N = \frac{1}{100} - \left(-\frac{1}{120}\right) = \frac{11}{600}$$

If L > OSD

From Eq. 4.33 length of summit curve,

$$L = \frac{N S^2}{9.6} = \frac{11 \times 470^2}{600 \times 9.6} = 422 \text{ m}$$

As this value is less than OSD of 470 m, assume L less than OSD.

If L > OSD

From equation 4.34 length of summit curve,

$$L = 2S - \frac{9.6}{N}$$

$$= 2 \times 470 - \frac{9.6 \times 600}{11} = 416.4 \text{ m, say } 417 \text{ m}$$

This value is less than 440 m.

Therefore, the length of summit curve = 417 m.

Example 4.24

A vertical summit curve is to be designed when two grades, + 1/50 and – 1/80 meet on a highway. The stopping sight distance and overtaking sight distance required are 180 and 640 m respectively. But due to site conditions the length of vertical curve has to be restricted to a maximum value of 500 m if possible. Calculate the length of summit curve needed to fulfil the requirements of (a) Stopping sight distance (b) Overtaking sight distance or atleast Intermediate sight distance and discuss the results.

Solution

$$N = +\frac{1}{50} - \left(-\frac{1}{80}\right) = \frac{13}{400}$$

(a) Requirements of stopping sight distance

$$SSD = 180 \text{ m}$$

Assume L > SSD

$$L = \frac{NS^2}{4.4} = \frac{13 \times 180}{400 \times 4.4} = 239.3 \text{ m}$$

As this length is greater than SSD the assumption is correct.

The length of summit curve required is 240 m which is less than the prescribed maximum limit of 500 m.

(b) Requirement of overtaking sight distance

$$OSD = 400 \text{ m}$$

Assume L > OSD

$$L = \frac{NS^2}{9.6} = \frac{13 \times 640^2}{400 \times 9.6} = 1387 \text{ m}$$

As the length of summit curve obtained is higher than the sight distance, length required is 1387 m.

As suggested in problem if the length of the summit curve is restricted to a value less than 500 m, it is not possible to provide the required O.S.D. of 640 m.

Therefore, to provide limited opportunities for overtaking, Intermediate Sight Distance (ISD) equal to twice the SSD of 180 × 2 = 360 m may be provided if possible.

If L > SD

$$L = \frac{NS^2}{9.6} = \frac{13 \times 360^2}{400 \times 9.6} = 439 \text{ m}$$

As this value is greater than the SD of 360 m, the assumption is correct. It is possible to provide the ISD of 439 m to allow limited overtaking operations and the length of summit curve in this case is less than the maximum available length of 500 m.

Example 4.25

A valley curve is formed by a descending grade of 1 in 25 meeting an ascending grade of 1 in 30. Design the length of valley curve to fulfil both comfort condition and head light sight distance requirements for a design speed of 80 kmph. Assume allowable rate of change of centrifugal acceleration $C = 0.6 \text{ m/sec}^3$.

Solution

$$N = -\frac{1}{25} - \frac{1}{30} = -\frac{11}{150}$$

$$V = 80 \text{ kmph}, v = 80/3.6 = 22.2 \text{ m/sec}$$

(i) Comfort Condition

From Eq. 4.34,

$$L = 2\left[\frac{Nv^3}{C}\right]^{\frac{1}{2}} = 2\left[\frac{11}{150} \times \frac{22.2^3}{0.6}\right]^{\frac{1}{2}} = 73.1 \text{ m}$$

(ii) Head Light Sight Distance Condition

Neglecting the ascending and descending gradients at the valley curve using Eq. 4.1 and assuming t = 2.5 secs. and f = 0.35, $SSD = v\,t + v^2/2gf = 22.2 \times 2.5 + (22.2^2/2 \times 9.8 \times 0.35) = 127.3$ m

If L > SSD, using Eq. 4.38

$$L = \frac{NS^2}{(1.5 + 0.035S)} = \frac{11 \times 127.3^2}{150(1.5 + 0.035 \times 127.3)} = 199.5 \text{ m}$$

As this value is higher than the SSD of 127.3 m, the assumption is correct. The valley curve length based on head light sight distance being higher than that based on comfort condition, the design length of valley curve is 199.5 or say, 200 m.

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PROBLEMS

1 What are the objects of highway geometric design ? List the various geometric elements to be considered in highway design.

2 Explain the role of pavement surface characteristics in highway geometric design. State the factors affecting friction between pavements and tyres of vehicles ?

3 Explain camber. What are the objects of camber ? Discuss the factors on which the amount of camber to be provided depends. Specify the recommended ranges of camber for different types of pavement surfaces.

4 Discuss the effects of shape of camber and the effects of providing steep cross fall.

5 Enumerate the factors governing the width of carriage way. State the IRC specifications for width of carriage way for various classes of roads.

6 Write short notes on :

(a) Traffic separators

(b) Kerbs

(c) Roads margins

(d) Pavement unevenness

(e) Shoulders

(f) Width of formation (or road way)

(g) Right of way.

7 Draw the typical cross sections of the following roads indicating the width of pavement, roadway and land

(a) M.D.R. in embankment

(b) O.D.R. in cutting

(c) National highway in embankment in rural area

(d) National highway in cutting

(e) A city road

(f) A divided highway in urban area

8 Explain sight distance and factors causing restrictions to sight distance. Explain the significance of stopping, intermediate and overtaking sight distances.

9 What are the factors on which the stopping sight distance depends ? Explain briefly ?

10 Explain total reaction time of driver and the factors on which it depends. Explain "PIEV" theory.

11 Derive an expression for finding the stopping sight distance at level and at grades.

12 Calculate the stopping sight distance for a design speed of 100 kmph. Take the total reaction time 2.5 second and the coefficient of friction = 0.35.

13 Find the stopping sight distance for a design speed of 65 kmph. Assume suitable data. What are sight distance requirements at a gradient of 1 in 40.

(*Hint for Solution* : Assume total reaction time as 2.5 second and design coefficient of friction as 0.36; find the stopping sight distances for level, ascending and descending grades).

14 State factors on which the overtaking sight distance depends. Explain briefly.

15 Derive an expression for calculating the overtaking sight distance on a highway.

16 Why are overtaking zones provided ? What is the basis of deciding its length ? Draw a neat sketch and show the signs to be installed and their positions.

17 The speeds of overtaking and overtaken vehicle are 80 and 60 kmph respectively. If the acceleration of the overtaking vehicle is 2.5 kmph per second, calculate the safe passing sight distance for

(a) one-way traffic

(b) two-way traffic

18 In problem 17, what should be the length of overtaking zone. Show the position of signs by means of a sketch.

19 Find the safe overtaking sight distance for a highway having a design speed of 100 kmph. Assume all data suitably.

[*Hint for Solution* : Assume speed of overtaken vehicle = (100 − 16) kmph; maximum acceleration of overtaking vehicle = 1.6 kmph/sec, as given in Table 4.6].

20 Discuss the factors to be considered in deciding the sight distance at intersections.

21 (a) Enumerate the design elements to be considered at the horizontal alignment.

(b) What are the effects of speed on horizontal alignment design ? What are design speeds for different classes of roads specified by the IRC ?

22 (a) Explain superelevation. What are the factors on which the design of superelevation depends ?

(b) Explain maximum and minimum superelevations.

23 Derive an equation for finding the superelevation required if the design coefficient of lateral friction is 'f'.

24 Enumerate the steps for practical design of superelevation ?

25 Design the superelevation required at a horizontal curve of radius 300 m for speed of 60 kmph. Assume suitable data.

26 Calculate the maximum allowable speed on a horizontal curve of radius 350 m if the maximum allowable values of lateral coefficient of friction is 0.15 and rate of superelevation is 0.07.

27 A radius of 250 m has to be provided at a locality due to site restrictions in a National Highway with design speed 100 kmph. Design the superelevation. Should there be restriction in speed ?

28 Explain with the aid of neat sketches the methods of eliminating camber and introduction of superelevation.

29 What is the basis of calculating absolute minimum radius of horizontal curve ? What do you understand by ruling minimum radius of curve ?

30 Calculate the absolute minimum and ruling minimum radius of horizontal curve for a design speed of 80 kmph.

31 State the objects of widening pavement on horizontal curves ? What are the factors on which the design of widening depends ? Explain.

32 Derive an expression for finding the extra widening required on horizontal curve.

33 (a) Why should the psychological widening be added to the mechanical widening ?

(b) How is the widening of pavements introduced in the field ?

(c) What is off-tracking ? Explain with sketches.

(d) Determine the off-tracking of a vehicle with wheel base 7.0 m while negotiating a horizontal curve of radius 100 m.

34 Calculating the extra width of pavement required on a horizontal curve of radius 700 m on a two lane highway, the design speed being 80 kmph. Assume wheel base l = 6 m.

35 In a mountainous terrain, a circular curve of radius 50 m and length 40 m has transition of 20 m on both ends. Calculate extra widening if the design speed is 30 kmph. Suggest suitable method of providing the widening on a two lane pavement.

36 What are the objects of providing transition curves on the horizontal alignment highways ? Explain.

37 List the various types of transition curves used in highways. What is an ideal transition curve ? Explain.

38 Discuss the factors to be considered while designing the length of transition curve.

39 Derive an expression for finding length of transition curve on horizontal alignment of highways.

40 The radius of a horizontal curve is 400 m, the total pavement width at curve is 7.6 m and the superelevation is 0.07. Design the transition curve length for a speed of 100 kmph. Assume pavement to be rotated about the inner edge.

41 Calculate the length of transition curve for a design speed of 80 kmph at horizontal curve of radius 300 m in a rural area. Assume suitable data.

(*Hint for Solution* : Solve the problem as in Example 4.16).

42 (a) Explain curve resistance and compensation in gradient on horizontal curves.

(b) There is a horizontal curve of radius 60 m on a stretch of hill road with a gradient of 5.0%. Determine the grade compensation.

43 Explain with sketches how sight distance is restricted on horizontal curves and how the desired sight distance could be obtained.

44 The stopping sight distance required for a highway is 80 m. Find the required set back distance from centre line of a circular curve of radius 300 m assuming the length of the curve is greater than the sight distance.

45 The overtaking sight distance required on a highway is 250 m. Find the required clearance of obstruction from centre line of a circular curve of radius 350 m and length 180 m. Assume two lane highway with d = 1.9 m.

46 A national highway passing through a flat terrain has a horizontal curve of radius equal to the ruling minimum radius. If the design speed is 100 kmph, calculate absolute minimum sight distance, superelevation, extra widening and length of transition curve. Assume necessary data suitably.

47 Enumerate the various design factors controlling the vertical alignment of highways.

48 (a) Explain ruling, maximum and exceptional gradients. Specify the values recommended by IRC for plains and hill.

(b) State the various considerations in deciding the ruling gradient of highway.

49 The ruling gradient of a hill road is 1 in 20. What should be the compensation in gradient and compensated gradient on a horizontal curve of radius 80 m after allowing for curve resistance ?

50 Explain summit and valley curves and the various cases when these are formed while two different gradients meet.

51 State the factors that govern the length of summit curve. How is it decided ?

52 Discuss the requirement of summit curves and its shape.

53 Explain how the vertical curves on a hump formed due to the presence of a culvert slightly above the profile may be designed.

54 Explain the factors based on which the length of valley curve is designed.

55 Discuss the problems in highway valley curves and the best shape of a valley curve.

56 A vertical summit curve is formed when an ascending gradient of 1 in 25 m another ascending gradient of 1 in 100. Find the length of the summit curve to provide the required stopping sight distance for a design speed of 80 kmph.

57 The deviation angle at a summit curve is 0.05 and the overtaking sight distance is 300 m. Find the length of summit curve required.

58 An ascending gradient of 1 in 50 meets a descending gradient of 1 in 80. Determine the length of summit curve to provide (a) ISD (b) OSD, for design speed of 80 kmph. Assume all other data.

59 A valley curve is formed by a descending gradient of 1 in 40 which meets an ascending gradient of 1 in 30.

(i) Design the total length of valley curve if the design speed is 100 kmph so as to fulfil both comfort condition and head light sight distance for night driving, after calculating the SSD required.

(ii) Find the position of the lowest point of the valley curve to locate a culvert.

Chapter 5
Traffic Engineering

5.1 INTRODUCTION

5.1.1 General

Traffic engineering is that branch of engineering which deals with the improvement of traffic performance of road networks and terminals. This is achieved by systematic traffic studies. Scientific analysis and engineering applications. The method includes planning and geometric design on one hand and regulation and control on the other. Traffic Engineering therefore deals with the application of scientific principles, tools, techniques and findings for safe, rapid, convenient and economic movement of people and goods.

The road traffic is composed of various categories of vehicular traffic and the pedestrian traffic. Each category of vehicular traffic has two components, the human element as the driver and his machine as vehicles. Traffic engineering has also to be recognized and governed by social and physical science. The profession of traffic engineering as known today has evolved with the advent of motor vehicle. During the last few decades significant advances have been made in many phases of the profession. Advanced study and training facilities have been made available at several universities and institution notably in the U.S.A. Traffic engineering has now been recognized as an essential tool in the improvement of traffic operations in metropolitan cities like Bombay, Delhi, Calcutta and Madras.

Definition

Institute of Traffic Engineers, U.S.A. defines, "Traffic engineering is that phase of engineering which deals with planning and geometric design of streets, highways, abutting lands, and with traffic operation thereon, as their use is related to the safe, convenient and economic transportation of persons and goods".

Professor Ress Blunden of California University has proposed a modified definition, "Traffic engineering is the science of measuring traffic and travel, the study of the basic laws relating to traffic flow and generation and application of this knowledge to the professional practice of planning, designing and operating traffic systems to achieve safe and efficient movement of persons and goods".

5.1.2 Scope of Traffic Engineering

The basic object of traffic engineering is to achieve efficient free and rapid flow of traffic, with least number of traffic accidents. Factual studies of traffic operations provide the foundation for developing methods for improvement in general and for solving specific problems.

The study of traffic engineering may be divided into six major sections, viz.:

(i) Traffic characteristics

(ii) Traffic studies and analysis

(iii) Traffic operation-control and regulation

(iv) Planning and analysis

(v) Geometric design

(vi) Administration and management

Study of traffic characteristics is the most essential prerequisite for any improvement of traffic facilities. The traffic characteristics are quite complex with various types of road users in the roads moving with different motives. The human psychology is to be given particular attention. The study of vehicular characteristics is an essential part. Apart from these the various studies to be carried out on the actual traffic include speed, volume, capacity, travel patterns, origin and destination, traffic flow characteristics; parking and accident studies.

Various aspects that are covered under traffic operations are regulations, control and the warrants for application of controls. Regulations may be in the form of laws and ordinances or other traffic regulatory measures such as speed limit etc. Installation of traffic control devices like signs, signals and islands are most common means to regulate and control the traffic. Actual adoption of traffic management measures, such as traffic regulations and control need adequate attention.

Traffic planning is a separate phase for major highways like *express-ways*, arterial roads, mass transit facilities, and parking facilities. All the aspects such as cross section and surface details, sight distance requirement, horizontal and vertical alignment, manoeuvre areas and intersections and parking facilities are to be suitably designed for better performance.

The various phases of traffic engineering are implemented with the help of *Engineering, Enforcement and Education* or "3-Es". Enforcement is usually made through traffic laws, regulations and control. Education may be possible by sufficient publicity and through schools and television. It aims at improving the human factor in traffic performance. Engineering phase is the one which is constructive. It deals with improvement of road geometrics, providing additional road facilities and installation of suitably designed traffic control devices.

5.2 TRAFFIC CHARACTERISTICS

5.2.1 Road User Characteristics

The human element is involved in all actions of the road users either as pedestrian, cyclist, cart driver or motorist. The physical, mental and emotional characteristics of human beings affect their ability to operate motor vehicle safely or to service as a pedestrian. Hence it is important to the traffic engineer to study the characteristics and limitations of the road users.

The various factors which affect road user characteristics may broadly be classified under four heads :

Physical

Mental

Psychological and

Environmental

Physical characteristics

The physical characteristics of the road users may be either permanent or temporary. The pavement characteristics are the vision, hearing, strength and the general reaction to traffic situations.

Vision plays the most important role of all these. These include the acuity of vision, peripheral vision and eye movement; glare vision, glare recovery and depth judgement. Minimum standards for acuity of vision are often laid down by licensing authorities. Field of clearest and acute vision is within a cone whose angle is only 3 degrees, though the vision is fairly satisfactory up to 10° in general and even upto 20° in the horizontal plane. However in the vertical plane the field of clear vision may be about two thirds of that in the horizontal plane. These factors are particularly taken care of while designing and installing control devices. As the field of clear vision is limited, the road users have to often shift their eyes within the peripheral field to obtain clear vision. The total time taken for the eye movement depends on some of the physical characteristics including the response to stimuli. The effects of glare, adoptability to changes of light i.e., darkness to light and bright light to darkness, should also be studied. The depth judgement is important for a driver in judging distance and speed of vehicles and other objects ahead.

Hearing helps drivers in a way, though it is more important for pedestrians and cyclists. Though strength is not an important factor in general, lack of strength may make parking maneouvers difficult, particularly for heavy vehicles. The reaction to traffic situations depends on the time required to perceive and understand the traffic situation and to take the appropriate action. This depends on many factors such as permanent and temporary physical factors mental and psychological set up, speed and environmental factors. Also the time required to take an appropriate action depends on the type of the problem and the familiarity. The PIEV theory explaining the total reaction time has already been discussed in Art. 4.3.2.

The temporary physical characteristics of the road users affecting their efficiency are fatigue, alcohol or drugs and illness. All these reduce alertness and increase the reaction time and also affect the quality of judgement in some situations.

Mental characteristics

Knowledge, skill, intelligence experience and literacy can affect the road user characteristics. Knowledge of vehicle characteristics, traffic behaviour, driving practice, rules of roads and psychology of road users will be quite useful for safe traffic operations. Reactions to certain traffic situations become more spontaneous with experience. Understanding the traffic regulation and special instruction and timely action depends on intelligence and literacy.

Psychological factors

These affect reaction to traffic situations of road users to a great extent. The emotional factors such as attentiveness, fear anger, superstition impatience, general attitude towards traffic and regulations and maturity also come under this. Distractions by non-traffic events and worries reduce attentiveness to traffic situations. Dangerous actions are likely due to impatience. Some road users do not pay due regard to the traffic regulations and do not have the right attitude towards the traffic.

Environmental factors

The various environmental conditions affecting the behaviour of road user are traffic stream characteristics, facilities to the traffic, atmospheric conditions and the locality. The traffic stream may consist of mixed traffic or heavy traffic whereas the facilities to overtake for faster vehicles may be limited. The adoptability to different traffic stream characteristics depends on the driver's characteristics as well as the motivation. The purpose of entering the traffic stream can be social, recreational, business, routine movement or an emergency dash. The time, place and route are fundamentally chosen by the road user based on the needs. Whatever be the motive of movement, once the individual enters the traffic stream, the road user, is usually motivated by the desire for time-distance economy on one hand, and comfort and safety on the other. Together with modifying factors of motivation, there is a great variation among road users and their behaviour in every traffic stream. The locality may be a shopping centre or a place with other distractions to the road users, thus affecting their behaviour. The other environmental factors of importance are the weather visibility and other atmospheric conditions.

The total reaction time or the "PIEV" time of the drivers vary considerably from driver to driver based on the above road users characteristics. But the reaction time of a particular driver may vary depending on the type of the problem and also environmental and modifying factors.

Driver characteristics such as the simple, reaction time, depth judgement, field of vision, visual acuity, glare recovery etc. may be measured in the laboratory using Driver Testing Equipment.

5.2.2 Vehicular Characteristics

It is quite important to study the various vehicular characteristics which affect the design and traffic performance, because it is possible to design a road for any vehicle but not for an indefinite vehicle. The basic criterion of highway engineering is to cater for the needs of existing and anticipated traffic. It will not be economically feasible to keep on increasing the geometric standards and thickness of pavements from time to time to meet the needs of a few vehicles whose dimensions and weight are increased. Hence the vehicle standards should be uniform at least within a country, keeping in view the large percentage of existing vehicles and those likely to be manufactured in the near future. The standards for the dimensions and weights of vehicles should be consistent with the road facilities now available or could be made available in the near future. The various vehicular characteristics affecting the road design may be classified as static and dynamic characteristic of the vehicles.

Static characteristics of vehicles affecting road design are the dimensions, weight and maximum turning angle. The height of vehicle affects the clearance of the overhead structures. The height of driver seat affects the visibility distance and the height of head light affects the head light sight distance at valley curves. The field of vision ahead for the driver also depends on the design of wind shield and the front portion of the vehicle body. The clearance below the chassis, approach, departure and ramp angles of the vehicle affects the design of verticle profile of drive ways, humps and dips. The length of vehicles affects the capacity, overtaking distance and maneuverability of vehicles. The minimum turning radius depends on the length of wheel base and the features of the steering system and this affects design of sharp curves for the manoeuvre of vehicles at slow speeds. Gross weight, axle and wheel loads of vehicle govern the structural design of pavements and cross drainage structures.

Dynamic characteristics of vehicles affecting road design are speed, acceleration and braking characteristics and some aspects of vehicle body design. The speed and acceleration depends upon the power of the engine and the resistances to be overcome and are important in all the geometric design elements. The deceleration and braking characteristics guide safe vehicle operation. The stability of vehicle and its safe movement on horizontal curves are affected by the width of wheel base and the height of centre of gravity. The riding comfort on vertical curves depends on the design of suspension system of the vehicle. The impact characteristics on collision and the injuries to the occupants depends on the design of the bumper and body of vehicle. Some of the vehicle characteristics have been discussed below in detail.

Vehicle dimensions

The dimensions to be mainly considered are the overall width, height, and length of different vehicles, particularly of the largest ones. The width of the vehicle affects the width of the traffic lanes, shoulders and parking facilities. If the width of the lanes are not adequate in view of the widest vehicles using the roads, the capacity of road will decrease. Height of the vehicle affects the clearance to be provided under structures such as overbridges underbridges, electric and other service lines. Length of the vehicle is an important factor in the design of horizontal alignment as it effects the extra width of pavement and minimum turning radius. Length affects the safe overtaking distance, capacity of a road and parking facilities. The length should also be considered in the design of valley curves and dips. The maximum allowable width, height and length of vehicles have been standardized by the Indian Roads Congress and are as given in Table 5.1 (a). The configurations of different types of transport vehicles and their axle arrangement are given in Fig. 5.1.

Table 5.1 (a) Maximum Dimensions of Road Vehicles

Dim. of Vehicle	Details	Maximum dim., m (excluding front & rear bumpers)
Width	All vehicles	2.50
Height	(a) Single-decked vehicle for normal application	3.80
	(b) Double-decked vehicle	4.75
Length	(a) Single-unit truck with two or more axles (types 2, 3)	11.00
	(b) Single-unit bus with two or more axles (types 2, 3)	12.00
	(c) Semi-trailer tractor combinations (Types 2-S1, 2-S2, 3-S1, 3-S2)	16.00
	(d) Tractor and trailer combinations (Types 2-2, 3-2, 2-3, 3-3)	18.00

No combination is allowed to be of more than two units, and no such combination, laden or unladen is allowed to have an overall length exceeding 18 m. The other dimensions to be considered are the wheel base (which affects the minimum turning

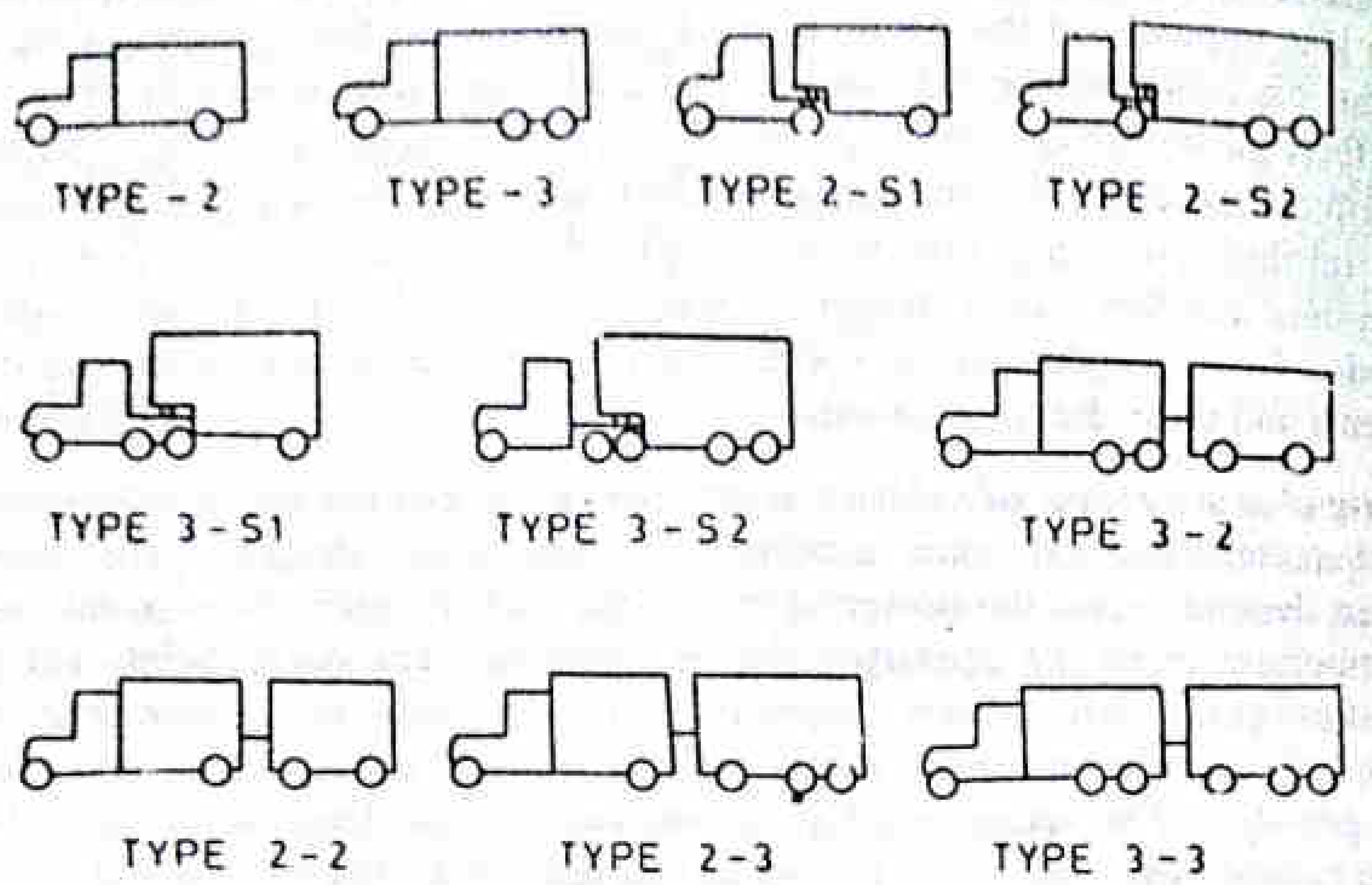


Fig. 5.1 Types of Road Transport Vehicles

radius and extra widening required) the front, rear and centre clearances and the approach, departure and ramp angles (which affect the design of vertical profile for highways and driveways).

Weight of loaded vehicle

The maximum weight of loaded vehicle affects the design of pavement thickness and gradients. In fact the limiting gradients are governed by both the weight and power of the heavy vehicles. The maximum permissible gross weights and axle weights have been standardized by IRC as per Table 5.1 (b).

Table 5.1 (b) Maximum Permissible Gross Weight and Axle Weight of Transport Vehicles

Vehicle type	Maximum Gross weight, tonnes	Maximum axle weight, tonnes			
		Truck/Tractor		Trailer	
		FAW	RAW	FAW	RAW
Type 2 (Both axles single tyre)	12.0	6	6.0		
Type 2 : (FA-Single tyre RA-Dual tyre)	16.0	6	10.2		
Type 3	24.0	6	18 (TA)		
Type 2-S1	26.4	6	10.2		10.2
Type 2-S2	34.2	6	10.2		18 (TA)
Type 3-S1	34.2	9	18 (TA)		10.2
Type 3-S2	42.0	6	18 (TA)		18 (TA)
Type 2-2	36.6	6	10.2	10.2	10.2
Type 3-2	44.4	6	18 (TA)	10.2	10.2
Type 2-3	44.4	6	10.2	10.2	18 (TA)
Type 3-3	52.2	6	18 (TA)	10.2	18 (TA)

FAW = Weight on Front Axle; RAW = Weight on Rear Axle

TA = Tandem axle, fitted with 8 tyres.

Power of vehicle

The power of the heaviest vehicles and their loaded weights govern the permissible and limiting values of gradient on roads. In this regard the total resistances to traction consisting of inertia, rolling resistance, air resistance and grade resistance are considered. From the total hauling capacity and the power required to overcome the total tractive resistance, it is possible to determine the speed and acceleration of the vehicle which in turn is useful in traffic regulation, planning, and design.

Speed of vehicle

The vehicle speed affects, (i) sight distances (ii) superelevation, length of transition curve and limiting radius on horizontal curves (iii) length of transition curves on vertical valley curves and on humps (iv) width of pavement and shoulders on straight and on horizontal curves (v) design gradient (vi) capacity of traffic lane (vii) design and control measures on intersections.

Thus the design speed controls most of the geometric features of highways.

Braking characteristics

The deceleration and braking characteristics of vehicles depend on the design and type of braking system (such as mechanical, fluid or air brake) and its efficiency. The safety of vehicle operation, stopping distance and the spacing between the two consecutive vehicles in a traffic stream are affected by the braking capacity. Thus the highway capacity and overtaking sight distance requirements also indirectly get affected.

Braking Test

It is possible to measure the skid resistance of pavement surface under the prevailing conditions by conducting braking tests on the road at the desired running speed. If the brakes are applied till the vehicle comes to stop, it may be assumed that wheels are fully locked and the brake efficiency is 100 percent. Atleast two of the following three measurements are needed during the braking tests in order to determine the skid resistance of the pavement :

(i) Braking distance, L metre

(ii) Initial speed, u m/sec.

(iii) Actual duration of brake application, t second

The method of calculating the average skid resistance of the pavement using two of the above three values has been illustrated with the help of Examples 5.1, 5.2 and 5.3.

Example 5.1

In a braking test, a vehicle traveling at a speed of 30 kmph was stopped by applying brakes fully and the skid marks were 5.8 m in length. Determine the average skid resistance of the pavement surface.

Solution

$$\text{Initial speed } u = \frac{30}{3.6} = 8.33 \text{ m/sec}$$

$$\text{Braking distance } L = 5.8 \text{ m} = \frac{u^2}{2gf} \quad \text{(see Eq. 4.1)}$$

$$\text{Average skid } f = \frac{8.33^2}{2 \times 9.8 \times 5.8} = 0.61$$

Example 5.2

A vehicle travelling at 40 kmph was stopped within 1.8 seconds after the application of the brakes. Determine the average skid resistance.

Solution

Initial speed $u = \frac{40}{3.6} = 11.11$ m/sec; Braking time t = 1.8 sec. Using the fundamental relation of motion for uniform acceleration/retardation,

$$v = u + at,\ v = 0,\ \text{retardation } a = \frac{u}{t} = \frac{11.11}{1.8} = 6.17 \text{ m/sec}^2$$

$$\text{From the relation, force } F = m\,a,\ W f = \frac{W a}{g}$$

$$\text{Average skid resistance } f = \frac{a}{g} = \frac{6.17}{9.88} = 0.63$$

Example 5.3

A vehicle was stopped in 1.4 second by fully jamming the brakes and the skid marks measured 7.0 m. Determine the average skid resistance.

Solution

Using the fundamental relations of motion for uniform acceleration/retardation

(i) $v = u + at$, as the final velocity $v = 0$, $u = -at$

(ii) $v^2 - u^2 = 2\,a\,s$

$$s = -\frac{u^2}{2a} = \frac{a^2 t^2}{2a} \text{ and therefore } a = \frac{2s}{t^2}$$

Given braking distance L = 7.0 m = s

and braking time t = 1.4 sec

$$\text{Average skid resistance } f = \frac{a}{g} = \frac{2 \times 7.0}{9.8 \times 1.4^2} = 0.729$$

Example 5.4

A vehicle moving at 40 kmph speed was stopped by applying the brake and the length of skid mark was 12.2 m. If the average skid resistance of the pavement is known to be 0.70, determine the brake efficiency of the test vehicle.

Solution

$$v = \frac{40}{3.6} = 11.11 \text{ m/sec},\ L = 12.2 \text{ m},\ f = 0.70$$

Average skid resistance developed

$$f' = \frac{v^2}{2\,gL} = \frac{11.11^2}{2 \times 9.8 \times 12.2} = 0.516$$

$$\text{Brake efficiency, \%} = \frac{100 f'}{f} = \frac{100 \times 0.516}{0.70} = 73.7\%$$

Off Tracking

When a four or six wheeled vehicle, such as car or bus (vehicle other than two and three wheelers) negotiates a horizontal curve at relatively slow speed, the rear wheels do not race the same path as the corresponding front wheels, as explained in Art. 4.4.6 under 'Mechanical Widening'. At relatively slow speeds when the centrifugal force developed is lesser than the counteracting forces due to the superelevation and transverse friction, the rear wheels follow paths on the inner side of the horizontal curve as compared with the path followed by the corresponding set of front wheels. This difference in distance between the curved wheel paths of a particular set of front and rear wheels (i.e., either the set of front and rear wheels on the outer side of the horizontal curve or the set on the inner side) is called off-tracking or the mechanical widening for a vehicle which is equal to $l^2/2R$ (see Eq. 4.15). Thus the off tracking depends on two factors :

(i) the length of wheel base or the distance between the front and real axles of the vehicle and

(ii) the turning angle or the mean radius of the horizontal curve traversed.

Example 5.5

A vehicle has a wheel base of 6.5 m. What is the off tracking while negotiating a curved path with a mean radius 32 m.

Solution

$$l = 6.5 \text{ m};\ R = 32 \text{ m}$$

$$\text{Off tracking} = \frac{l^2}{2R} = \frac{6.5^2}{2 \times 32} = 0.66 \text{ m}$$

5.2.3 Traffic Studies

Traffic studies or surveys are carried out to analyse the traffic characteristics. These studies help in deciding the geometric design feature and traffic control for safe and efficient traffic movements. The traffic surveys for collecting traffic data are also called *traffic census*.

The various traffic studies generally carried out are :

(a) Traffic volume study

(b) Speed studies

(i) spot speed study

(ii) speed and delay study

(c) Origin and destination (O & D) study

(d) Traffic flow characteristics

(e) Traffic capacity study

(f) Parking study

(g) Accident studies or the traffic flop

Traffic volume study

Traffic volume is the number of vehicles crossing a section of road per unit time at any selected period. Traffic volume is used as a quantity measure of flow; the commonly used units are vehicles per day and vehicles per hour. A complete traffic volume study may include the classified volume study by recording the volume of various types and classes of traffic, the distribution by direction and turning movements and the distribution on different lanes per unit time. The objects and uses of traffic volume studies are given below :

(a) Traffic volume is generally accepted as a true measure of the relative importance of roads and in deciding the priority for improvement and expansion.

(b) Traffic volume study is used in planning, traffic operation and control of existing facilities and also for planning and designing the new facilities.

(c) This study is used in the analysis of traffic patterns and trends.

(d) Classified volume study is useful in structural design of pavements, in geometric design and in computing roadway capacity.

(e) Volume distribution study is used in planning one-way streets and other regulatory measures.

(f) Turning movement study is used in the design of intersections, in planning signal timings, channelization and other control devices.

(g) Pedestrian traffic volume study is used for planning side walks, cross walks subways and pedestrian signals.

There are variations in traffic flow from time to time. Hourly traffic volume varies considerably during a day, the peak hourly volume may be much higher than average hourly volume. Daily traffic volumes vary considerably in a week and there are variations with season. Hence if a true picture is to be obtained, the hourly traffic volume should be known along with the patterns of hourly, daily and seasonal variations. In classified traffic volume study, the traffic is classified and the volume of each class of traffic viz., buses, truck, passenger-cars, other light vehicles, rickshaws, tongas, bullock carts, cycles and pedestrians is found separately. The direction of each class of traffic flow is also noted. At intersections the traffic flow in each direction of flow including turning movements are recorded.

Counting of traffic volume

Traffic volume counts may be done by mechanical counters or manually.

Mechanical counters

These may be either fixed (permanent) type or portable type. The mechanical counter can automatically record the total number of vehicles crossing a section of the road in a desired period. The working may be by the effect of impulses or stimuli caused by traffic

movements on a pneumatic hose placed across the roadway or by using any other type of sensor. Traffic count is recorded by electrically operated counters and recorders capable of recording the impulses. The impulses caused by vehicles of light weight may not be enough in some cases to actuate the counter. Also it is not possible to easily record pedestrian traffic by this method. Other methods of working the mechanical detectors are by *photo-electric cells, magnetic detector* and *radar detectors*. The main advantage of mechanical counter is that it can work throughout the day and night for the desired period, recording the total hourly volume, which may not be practicable in manual counting. The main drawback of the mechanical counter is that it is not possible to get the traffic volumes of various classes of traffic in the stream and the details of turning movements.

Manual counts

This method employs a field team to record traffic volume on the prescribed record sheets. By this method it is possible to obtain data which can not be collected by mechanical counters, such as vehicle classification, turning movements and counts where the loading conditions or number of occupants are required. However it is not practicable to have manual count for all the 24 hours of the day and on all days round the year. Hence it is necessary to resort to statistical sampling techniques in order to cut down the manual hours involved in taking complete counts, First the fluctuations of traffic volume during the hours of the day and the daily variations are observed. Then by selecting typical short count periods, the traffic volume study is made by manual counting. Then by statistical analysis the peak hourly traffic volumes as well as the average daily traffic volumes are calculated. This method is very commonly adopted due to the specific advantages over other methods.

Presentation of traffic volume data

The data colleted during the traffic volume studies are sorted out and are presented in any of the following forms depending upon the requirements.

(a) *Annual average daily traffic* (AADT or ADT) of the total traffic as well as classified traffic are calculated. This helps in deciding the relative importance of a route and in phasing the road development programme. In order to convert the different vehicle classes to one class such as passenger car, conversion factors known as *Passenger Car Units* (PCU) are used. (see Tables 5.2 and 5.3).

(b) Trend charts showing volume trends over period of years are prepared. These data are useful for planning future expansion, design and regulation.

(c) Variation charts showing hourly, daily and seasonal variations are also prepared. These help in deciding the facilities and regulation needed during peak traffic periods.

(d) Traffic flow maps along the routes, (the thickness of the lines representing the traffic volume to any desired scale), are drawn. These help to find the traffic volume distribution at a glance.

(e) Volume flow diagram at intersections either drawn to a certain scale or indicating traffic volume as shown in Fig. 5.2 are prepared, thus showing the details of crossing and turning traffic. These data are needed for intersection design.

(f) *Thirtieth highest hourly volume* or the design hourly volume is found from the plot between hourly volume and the number of hours in an year that the traffic volume is exceeded. See Fig. 5.3. The 30th highest hourly volume is the hourly volume that will be exceeded only 29 times in a year and all other hourly volumes of the year will be less

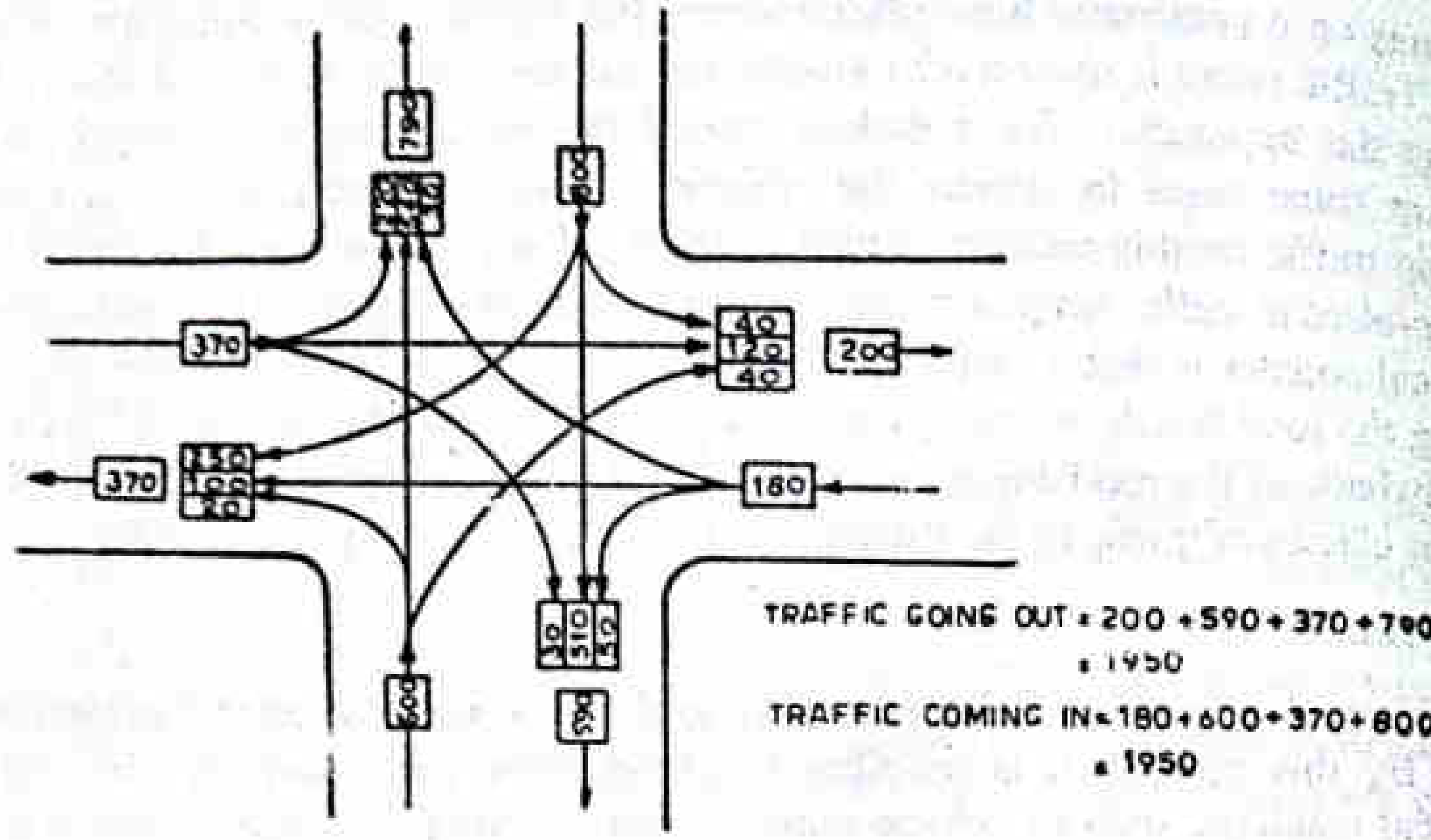


Fig. 5.2 Traffic Flow at Intersection

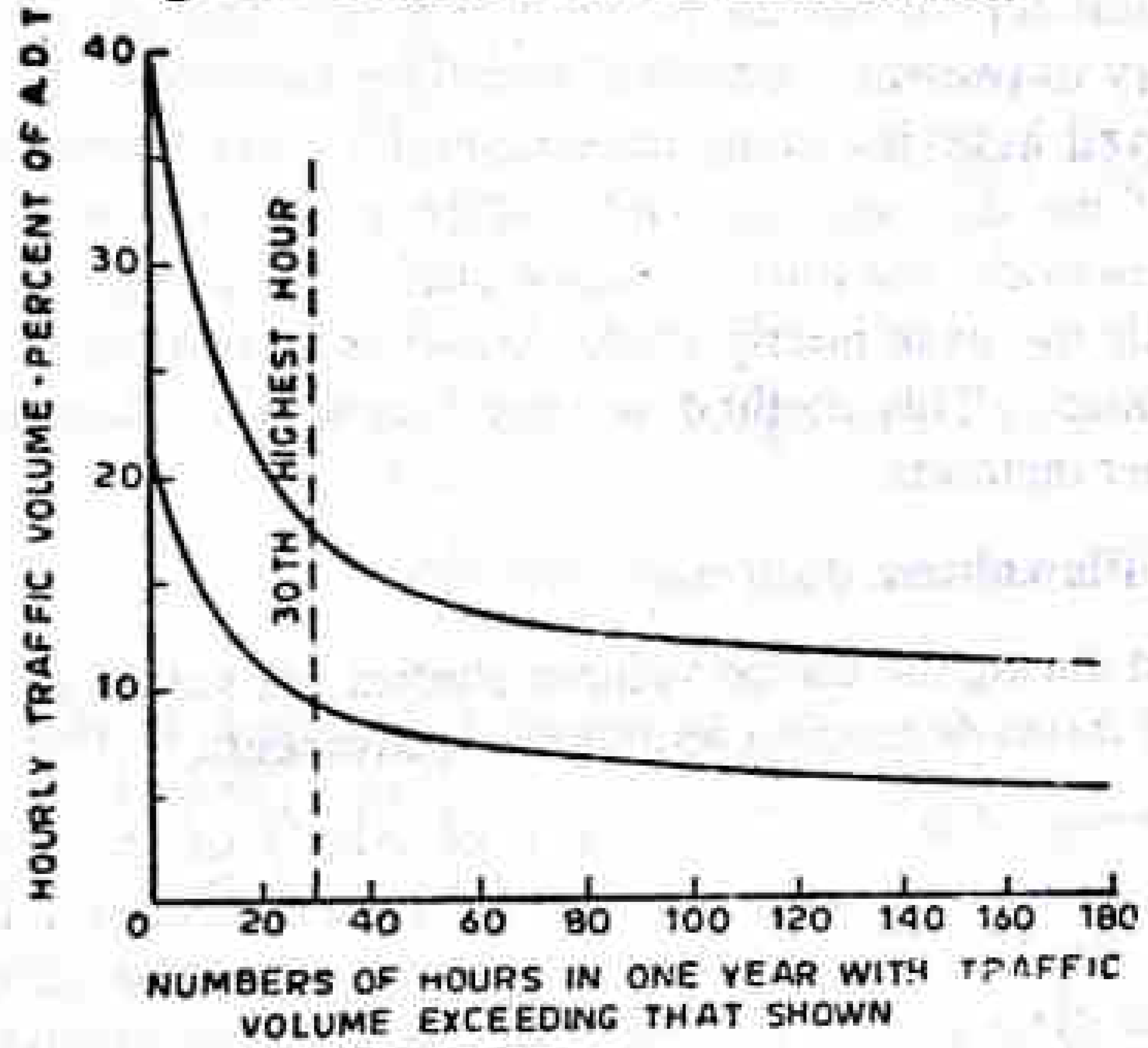


Fig. 5.3 Hourly Traffic Volumes

than this value. The highest or peak hourly volume of the year will be too high that it will not be economical to design the facilities according to this volume. The annual average hourly volume (AAHV) found from AADT will not be sufficient during considerable period of an year. The high facilities designed with capacity for 30th highest hourly traffic volume in the assumed year is found to be satisfactory from both facility and economic considerations. This is because the cost will be less when compared to the peak hourly volume and hence reasonable. There will be congestion only during 29 hours in the year. Thus the thirtieth highest hourly volume is generally taken as the hourly volume for design. However the actual design hourly volume may be decided by drawing the diagram as shown in Fig. 5.3, after carrying out traffic volume studies on the desired location of the road. The design hourly volume thus arrived at need not necessarily be the thirtieth highest hourly volume in all the cases.

Speed studies

The actual speed of vehicles over a particular route may fluctuate widely depending on several factors such as geometric features, traffic conditions, time, place, environment and driver.

Travel time is the reciprocal of speed and is a simple measure of how all well a road network is operating.

Spot speed is the instantaneous speed of a vehicle at a specified section or location.

Average speed is the average of the spot speeds of all vehicles passing a given point on the highway.

There are two definitions for the average of a series of spot speed measurements viz.; *space-mean speed* and *time-mean speed.* Space-mean speed represents the average speed of vehicles in a certain road length at any time. This is obtained from the observed t ravel time of the vehicles over a relatively long stretch of the road. Space-mean speed is calculated from :

$$V_s = \frac{3.6\,d\,n}{\sum_{i=1}^{n} t_i}$$

where V_s = space-mean speed, kmph

d = length of road, considered, m

n = number of individual vehicle observations

t_i = observed travel time (sec) for ith vehicle to travel distance d, m

The average travel time of all the vehicles is obtained from the reciprocal of space-mean speed.

Time-mean speed represents the speed distribution of vehicles at a point on the roadway and it is the average of instantaneous speeds of observed vehicles at the spot. Time-mean speed is calculated from :

$$V_t = \frac{\sum_{i=1}^{n} V_i}{n}$$

where V_t = time-mean speed, kmph

V_i = observed instantaneous speed of ith vehicles, kmph

n = number of vehicles observed

The space-mean speed is slightly lower than time-mean speed under typical speed conditions on rural highways.

Running speed is the average speed maintained by a vehicle over a particular stretch of road, while the vehicle is in motion; this is obtained by dividing the distance covered by the time during which the vehicle is actually in motion.

Overall speed or travel speed is the effective speed with which a vehicle traverses a particular route between two terminals; this is obtained by dividing the total distance travelled by the total time taken including all delays and stoppages enroute.

Speed studies carried out occasionally give the general trend in speeds. There are two types of speed studies carried out,

(i) Spot speed study, and

(ii) Speed and delay study

Spot speed study

Spot speed study may be useful in any of the following aspects of traffic engineering :

(a) to use in planning traffic control and in traffic regulations.

(b) to use in geometric design-for redesigning existing highways or for deciding design speed for new facilities.

(c) to use in accident studies

(d) to study the traffic capacity

(e) to decide the speed trends

(f) to compare diverse types of drivers and vehicles under specified conditions.

The spot speeds are affected by physical features of the road like pavement width, curve, sight distance, gradient, pavement unevenness intersections and road side developments. Other factors affecting spot speeds are environmental conditions (like weather, visibility), enforcement, traffic conditions, driver, vehicle and motive of travel.

There are a number of methods to measure spot speed. The spot speed may be obtained either by finding the running speed of vehicles over a short distance of less than 50 metre or by finding the instantaneous speed while crossing a section, depending on the method used. The spot speeds of a few typical sample of vehicles are found during the sampling periods of the day, days of the week and months of the year.

One of the simplest methods of finding spot speed is by using *enoscope* which is just a mirror box supported on a tripod stand. In its simplest principle, the observer is stationed on one side of the road and starts a stopwatch when a vehicle crosses that section. An enoscope is placed at a convenient distance of say 30 m in such a way that the image of the vehicle is seen by the observer when the vehicle crosses the section where the enoscope is fixed (see Fig. 5.4) and at this instant the stop watch is stopped. Thus the time required for the vehicle to cross the known length is found and is converted to the speed in kmph. The main advantage of this method is that it is a simple and cheap equipment and is easy to use. The greatest disadvantage is that the progress is so slow as it is difficult to spot out typical vehicles and the number of samples observed will be less. There is also a possibility of human error.

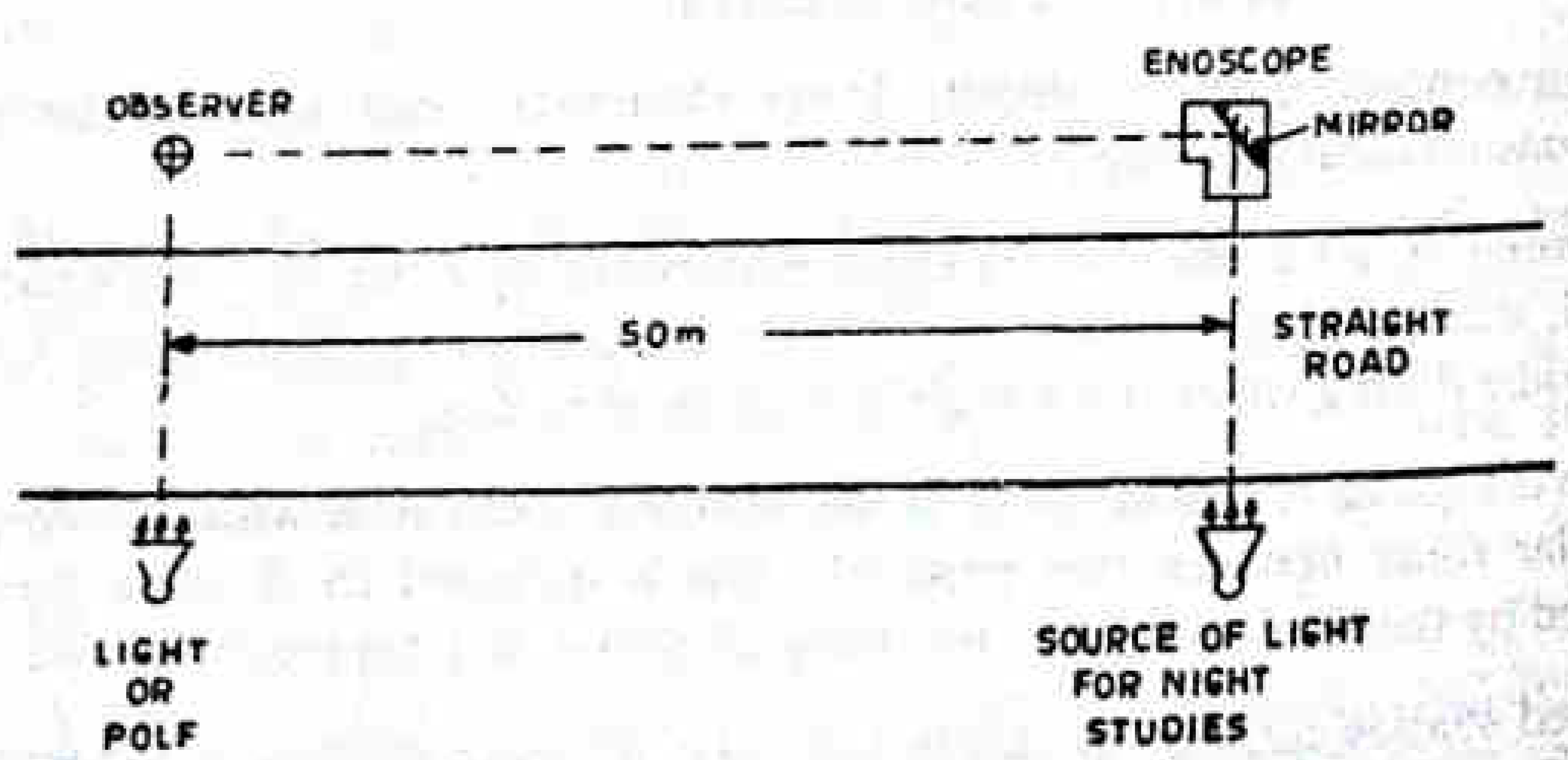


Fig. 5.4 Spot Speed by Enoscope

Other equipment used for spot speed measurements are graphic recorder, electronic meter, photo electric meter, radar, speed meter and by photographic methods. Of all these methods, the radar speed meter method seems to be the most efficient one as it is capable of measuring the spot speeds instantaneously and also record them automatically. But this equipment is costly.

Presentation of spot speed data

(a) *Average speed of vehicles* : From the spot speed data of the selected samples, frequency distribution tables are prepared by arranging the data in groups covering various speed ranges and the number of vehicles in such range. The *arithmetic mean* is taken as the average speed. The table gives the general information of the speeds maintained on the section, and also regarding the speed distribution pattern.

(b) *Cumulative speed of vehicles* : A graph is plotted with the average values of each speed group on the X-axis and the cumulative percent of vehicles travelled at or below the different speeds on the Y-axis. From this graph, the 85th *percentile speed* is found out which gives that speed at or below 85 percent of the vehicles are passing the point on the highway or only 15 per cent of the vehicles exceed the speed at that spot. See Fig. 5.5. The drivers exceeding 85th percentile speed are usually considered to drive faster than the safe speed under existing conditions and hence this speed is adopted for the *safe speed limit* at this zone. However for the purpose of highway geometric design, the 98th percentile speed is taken.

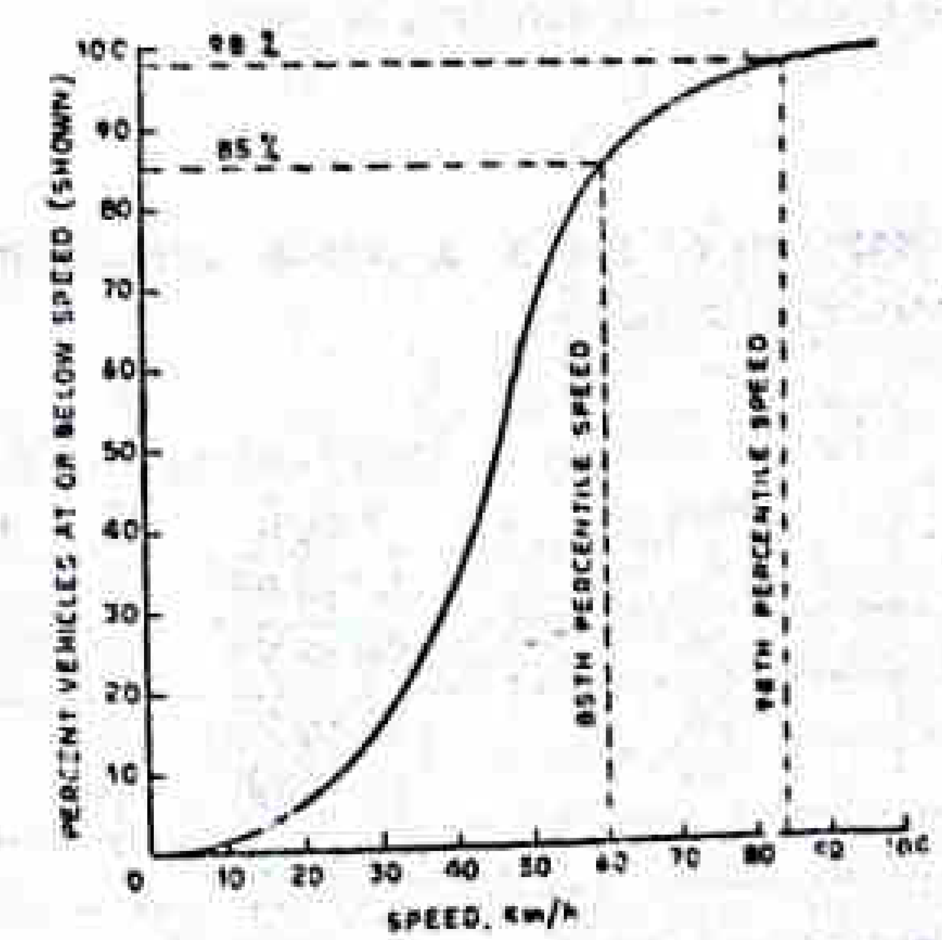


Fig. 5.5 Cumulative Speed Distribution

The 15th percentile speed represents the lower speed limit if it is desired to prohibit slow moving vehicles to decrease delay and congestion, as 85 percent of the vehicles to the stream travel at speeds higher than this value and therefore need overtaking opportunities.

(c) *Modal average* : A frequency distribution curve of spot speeds is plotted with speed of vehicles or average values of each speed group of vehicles on the X-axis and the percentage of vehicles in that group on the Y-axis. See Fig. 5.6. This graph is called the speed distribution curve. This curve will have a definite peak value of travel speed across the section and this speed is denoted as model speed. The speed distribution curve is helpful in determining the speed at which the greatest proportion of vehicles move, given by the model speed.

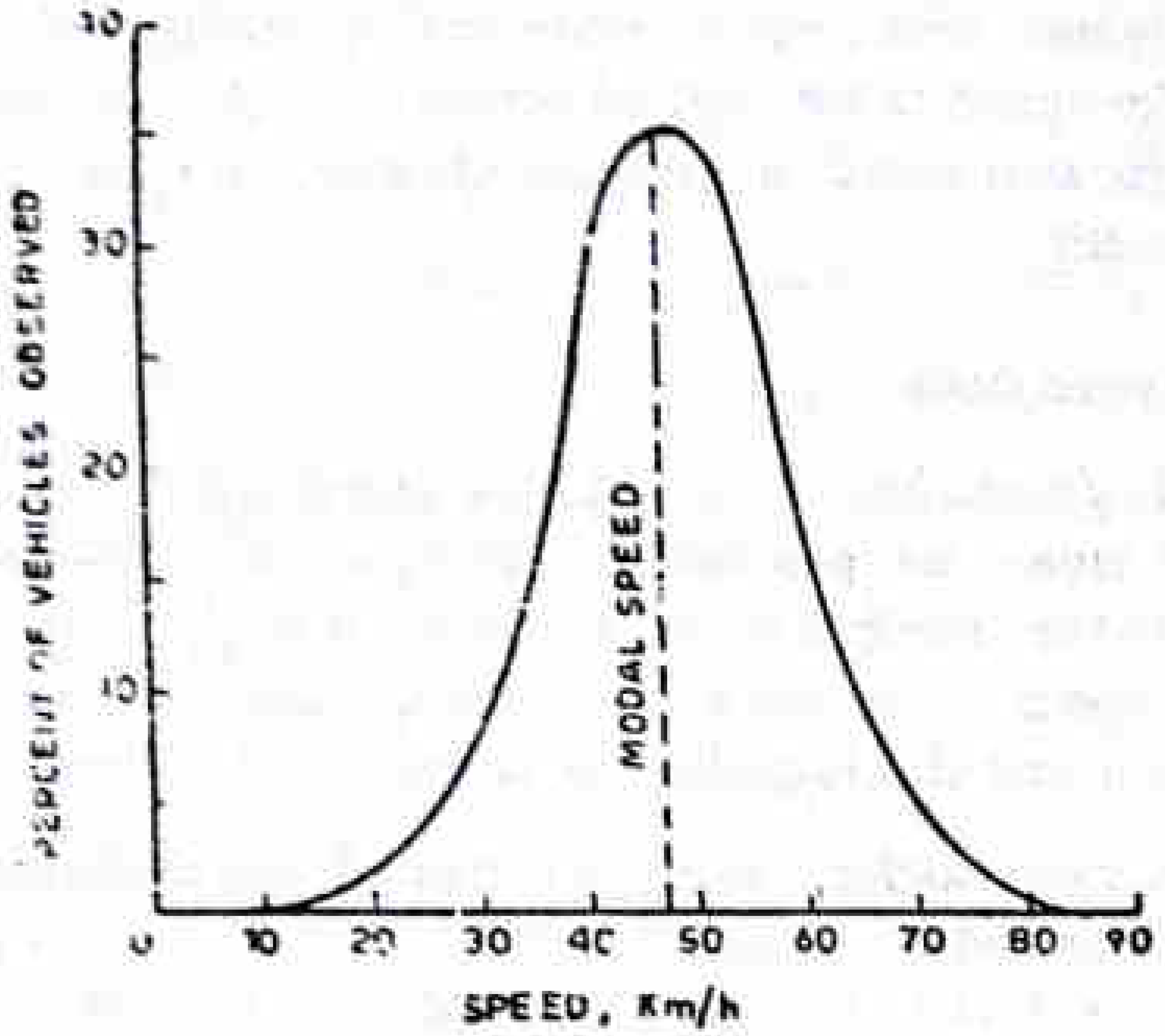


Fig. 5.6 Frequency Distribution Curve of the Spot Speeds

All vehicles do not travel at the same speed at a location along a road. The amount of speed dispersion or the spread from the average speed affects both capacity and safety. For free flow of vehicles, the speed distribution follows a normal distribution curve. The quality of flow of vehicles in a stream therefore depends on the speed dispersion. This may be judged by several methods such as (85^{th} minus 15^{th} percentile speeds); Standard deviation of speeds, or the coefficient of variation in speed.

Example 5.6

Spot speed studies were carried out at a certain stretch of a highway and the consolidated data collected are given below.

Speed range, Kmph	No. of vehicles observed	Speed range, kmph	No. of vehicles observed
0 to 10	12	50 to 60	255
10 to 20	18	60 to 70	119
20 to 30	68	70 to 80	43
30 to 40	89	80 to 90	33
40 to 50	204	90 to 100	9

Determine (i) the upper and lower values or speed limits for regulation of mixed traffic flow and (ii) the design speed for checking the geometric design elements of the highway.

Solution

The problem may be solved in three stages. First a frequency distribution table is prepared, next the cumulative frequency distribution curve is drawn and finally the appropriate values are obtained from the curve. Refer table 5.2. Column no.2 represents the average values of the different speed ranges. The number of vehicles observed in each speed range is represented as the frequency f in column no.3. The percentage frequency values given in Column no. 4 are based on the total number of vehicles observed in all the speed ranges. The cumulative values of percentage frequency are given in column no.5.

Table 5.2 Frequency Distribution of Spot Speed Data (Example 5.6)

Speed range, Kmph	Mid speed, kmph	Frequency, f	Frequency, %	Cumulative Frequency, %
1	2	3	4	5
0 – 10	5	12	1.41	1.41
10 – 20	15	18	2.12	3.53
20 – 30	25	68	8.00	11.53
30 – 40	35	89	10.47	22.00
40 – 50	45	204	24.00	46.00
50 – 60	55	255	33.00	76.00
60 – 70	65	119	14.00	90.00
70 – 80	75	43	5.06	95.06
80 – 90	85	33	3.88	98.94
90 – 100	95	9	1.06	100.00
	Total :	850	100.00	

Using the values of mid-speed and cumulative frequency % column 2 and 5 of Table 5.2, cumulative speed distribution curve is plotted (see Fig. 5.5); from this graph the following results are obtained :

(i) Upper speed limit for regulation = 85^{th} percentile speed

= 60 kmph

(ii) Lower speed limit for regulation = 15^{th} percentile speed

= 30 kmph

(iii) Speed to check design elements = 98^{th} percentile speed

= 84 kmph

Example 5.7

The table below gives the consolidated data of spot speed studies on a section of a road. Determine the most preferred speed at which maximum proportion of vehicles travels.

Speed range, kmph	No. of speed observations	Speed range, kmph	No. of speed observations
0 – 10	0	50 – 60	216
10 – 20	11	60 – 70	68
20 – 30	30	70 – 80	24
30 – 40	105	80 – 90	0
40 – 50	233		

Solution

The most preferred speed at which maximum proportion of vehicles travel is the modal speed which can be obtained by plotting the frequency distribution curve. The frequency distribution table of spot speeds prepared for this purpose is given in Table 5.3.

The frequency distribution curve is plotted (see Fig. 5.6) using the mean speed and the percent frequency values of the Table 5.3. The modal speed corresponds to the maximum value of percentage frequency in Fig. 5.6 and is equal to 47 kmph.

Table 5.3 Frequency Distribution of Spot Speed Data (Example 5.7)

Speed range, kmph	Mean speed, kmph	Frequency, f	Percent frequency
0 – 10	5	0	0.0
10 – 20	15	11	1.6
20 – 30	25	30	4.4
30 – 40	35	105	15.3
40 – 50	45	233	33.9
50 – 60	55	216	31.4
60 – 70	65	68	9.9
70 – 80	75	24	3.5
80 – 90	85	0	0.0
	Total :	687	105.0

Speed and delay study

The speed and delay studies give the running speeds, overall speeds, fluctuations in speeds and the delay between two stations of a road spaced far apart. They also give the information such as the amount, location, duration frequency and causes of the delay in the traffic stream. The results of the speed and delay studies are useful in detecting the spots of congestion, the causes and in arriving at a suitable remedial measures. The studies are also utilised in finding the travel time and in *benefit-cost analysis*. In general the efficiency of the roadway be judged from the travel time.

The delay or the time lost by traffic during the travel period may be either due to fixed delays or operational delays. Fixed delay occurs primarily at intersections due to traffic signals and at level crossings. Operational delays are caused by the interference of traffic movements, such as turning vehicles, parking and imparking vehicles, pedestrians etc., and by internal friction in the traffic stream due to high traffic volume, insufficient capacity and by accidents. Therefore the overall travel speed between the origin and destination points of travel is invariably lower than the desired running speed.

There are various methods of carrying out speed and delay study, namely :

Floating car or riding check method

License plate or vehicle number method

Interview technique

Elevated observations, and

Photographic technique

In the *floating car method* a test vehicle is driven over a given course of travel at approximately the average speed of the stream, thus trying to *float* with the traffic stream. A number of test runs are made along the study stretch and a group of observers record the various details. One observer is seated in the floating car with two stop watches. One of the stop watches is used to record the time at various control points like intersections, bridges or any other fixed points in each trip. The other stop watch is used to find the duration of individual delays. The time, location and cause of these delays are recorded by the second observer either on suitable tabular forms or by voice recording equipment. The number of vehicle overtaking the test vehicle and that overtaken by the test vehicles are noted in each trip by a third observer. The number of vehicles travelling in the

opposite direction in each trip is noted by a fourth observer. However in mixed traffic flow, more number of observers will be required to count the vehicles of different classes. In this method the detailed information is obtained concerning all phases of speed and delay including location, duration and causes of delay.

The average journey time $\bar{t}$ (minute) for all the vehicles in a traffic stream in the direction of flow q is given by :

$$\bar{t} = t_w - n_y/q \qquad (5.1)$$

$$q = \frac{n_a + n_y}{t_a + t_w} \qquad (5.2)$$

where

q = flow of vehicles (volume per min), in one direction of the stream

n_a = average number of vehicles counted in the direction of the stream when the test vehicle travels in the opposite direction

n_y = the average number of vehicles overtaking the test vehicle minus the number of vehicles overtaken when the test is in the direction of q

t_w = average journey time, in minute when the test vehicle is travelling with the stream q

t_a = average journey time, in minute when test vehicle is running against the stream q

In the *license plate or vehicle number method*, synchronized stop watches or voice recording equipment are used. Observers are stationed at the entrance and exit of a test section where information of travel time is required. The timings and the vehicle numbers are noted by the observers of the selected sample. From the office computations, travel time of each vehicles could be found. But the method does not give important details such as causes of delays and the duration and number of delays within the test section.

In the *interview technique*, the work can be completed in a short time by interviewing and collecting details from the road users on the spot. However the data collected may not provide with all the details correctly.

Elevated observation and *photographic technique* are useful for studying short test sections like intersection etc.

Intersection delays studies need special attention as this poses a major problem to the traffic engineer. Such studies at each intersection will help in evaluating the efficiency and effectiveness of the control device like signal system, the remedial measures for accidents etc.

Example 5.8

The consolidated data collected from speed and delay studies by floating car method on a stretch of urban road of length 3.5 km, running North-South are given below. Determine the average values of volume, journey speed and running speed of the traffic stream along either direction.

Trip No.	Direction of trip	Journey time Min. Sec.	Total stopped delay Min. Sec.	No. of vehicles overtaking	No. of vehicles overtaken	No. of vehicles from opp. direction
1	N – S	6 – 32	1 – 40	4	7	268
2	S – N	7 – 14	1 – 50	5	3	186
3	N – S	6 – 50	1 – 30	5	3	280
4	S – N	7 – 40	2 – 00	2	1	200
5	N – S	6 – 10	1 – 10	3	5	250
6	S – N	8 – 00	2 – 22	2	2	170
7	N – S	6 – 28	1 – 40	2	5	290
8	S – N	7 – 30	1 – 40	3	2	160

Solution

The mean values of journey time, stopped delay, number of vehicles overtaking, overtaken and in opposite direction for North-South and South-North directions are obtained from Table 5.4.

Table 5.4 Mean values of speed and Delay Data (Example 5.8)

Direction	Journey time Min. Sec.	Stopped delay Min. Sec.	Number of vehicles		
			Overtaking	Overtaken	In opposite direction
N – S	6 – 32	1 – 40	4	7	268
	6 – 50	1 – 30	5	3	280
	6 – 10	1 – 10	3	5	250
	6 – 28	1 – 40	2	5	290
Total :	26 – 00	6 – 00	14	20	1088
Mean :	6 – 30	1 – 30	3.5	5.0	272
S – N	7 – 14	1 – 50	5	3	186
	7 – 40	2 – 00	2	1	200
	8 – 00	2 – 22	2	2	170
	7 – 30	1 – 40	3	2	160
Total :	30 – 24	7 – 12	12	8	716
Mean :	7 – 36	! – 40	3.0	2.0	179

(i) North-South direction

n_y = average no. of vehicles overtaking minus overtaken = 3.5 – 5.0 = – 1.5

n_a = average no. of vehicles during trips in opposite direction (from S – N trips) = 179

t_w = average journey time = 6 min. 30 sec. = 6.5 min

t_a = average journey time during trips against the stream = 7 min. 36 sec. = 7.6 min

$$q = \text{average volume} = \frac{a_a + n_y}{t_a + t_w} = \frac{179 - 1.5}{7.6 + 6.5} = 12.59 \text{ veh/min}$$

$$\bar{t} = \text{average journey time} = t_w - \frac{n_y}{q} = 6.5 - \frac{(-1.5)}{12.59} = 6.62 \text{ min}$$

$$\text{Average journey speed} = \frac{3.5}{6.62} \text{ km/min} = \frac{3.5 \times 60}{6.62} = 31.7 \text{ kmph}$$

$$\text{Average stopped delay} = 1.5 \text{ min}$$

$$\text{Average running time} = \text{Average journey time} - \text{average stopped delay}$$

$$= 6.621 - 1.50 = 5.12 \text{ min}$$

$$\text{Average running speed} = \frac{3.5 \times 60}{5.12} = 41.0 \text{ kmph}$$

(ii) South-North direction

$$n_y = 3.0 - 2.0 = 1.0$$

$$t_w = 7.6 \text{ min}$$

$$t_a = 6.5 \text{ min}$$

$$n_a = (\text{from N – S strips}) = 272$$

$$q = \frac{272 + 1.0}{6.5 + 7.6} = 19.36 \text{ veh/min}$$

$$\bar{t} = 7.6 - \frac{1.0}{19.36} = 7.55 \text{ min}$$

$$\text{Journey speed} = \frac{3.5 \times 60}{7.55} = 27.8 \text{ kmph}$$

$$\text{Average stopped delay} = 1.8 \text{ min}$$

$$\text{Average running time} = 7.55 - 1.80 = 5.75 \text{ min}$$

$$\text{Average running speed} = \frac{3.5 \times 60}{5.75} = 36.5 \text{ kmph}$$

Origin and Destination Studies

The origin and destination (O & D) study is carried out mainly to (i) plan the road network and other facilities for vehicular traffic and (ii) plan the schedule of different modes of transportation for the trip demand of commuters.

The O & D studies of vehicular traffic determines their number, their origin and destination in each zone under study. The data may also be supplemented by the number of passengers in each vehicle, purpose of each trip, intermediate stops made and reasons etc. Origin and destination study gives informations like the actual direction of travel, selection of routes and length of the trip. These studies are most essential in planning new highway facilities and in improving some of the existing systems. As an example there can be a high percentage of through traffic which may be diverted by providing a by-pass and thus considerable saving in distance and time can be made. O & D study provides the basic data for determining the desired directions of flow or the *desire lines*. This is considered to be one of the important traffic studies needed to solve many traffic problems in a zone and the most important study to plan the highway system in a region.

Scientific planning of transportation system and mass transit facilities in cities should be based on O & D data of passenger trips. Also future traffic needs may by estimated by extrapolating the data from O & D study, together with socio-economic studies. (See Art. 5.2.5 for estimation of future traffic).

The various applications of O & D studies may be summed up as follows :

(i) to judge the adequacy of existing routes and to use in planning new network of roads.

(ii) to plan transportation system and mass transit facilities in cities including routes and schedules of operation.

(iii) to locate *expressway* or major routes along the desire lines.

(iv) to establish preferential routes for various categories of vehicle including by-pass.

(v) to locate terminals and to plan terminal facilities.

(vi) to locate new bridge as per traffic demands.

(vii) to locate intermediate stops of public transport.

(viii) to establish design standards for the road, bridges and culverts along the route.

There are a number of methods for collecting the O & D data. Some of the methods, commonly adopted are :

Road-side interview method, License plate method

Return post card method, Tag-on-car method and

Home interview method.

The choice of the method is made judiciously depending on the objective and location.

Road side interview method

The vehicles are stopped at previously decided interview stations, by a group of persons and the answers to prescribed questionnaire are collected on the spot. The information collected include the place and time of origin and destination, route, locations of stoppages, the purpose of the trip, type of vehicle and numbers of passengers in each vehicle. The traffic may be filtered through a prescribed lane by previous warning signs and with the help of police so that each driver of the selected sample of vehicles is interviewed. The percentage of sample interviewed out of the total traffic in each selected period should also be noted from appropriate traffic volume study taken simultaneously.

In this method the data is collected quickly in short duration and the field organisation is simple and the team can be trained quickly. The main drawback of the method is that the vehicles are stopped for interview, and there is delay to the vehicular movement. Also resentment is likely from the road users. Further, unless there is enough space, undue congestion may result due to stopped vehicles.

License plate method

The entire area under study is cordoned out and the observers are simultaneously stationed at all points of entry and exit on all the routes leading to and out of the area. Each party at the observation station is given synchronized time pieces and they note the license plate numbers (registration numbers) of the vehicles entering and leaving the cordoned area and the time. Separate recording sheets are maintained for each direction of movement for a specified time interval. After collecting the field data major work remains of the office computations and analysis, by tracking each vehicle number and its time of entering and leaving the cordoned area.

This method is quite easy and quick as far as the field work is concerned. The field organisation can also be trained quickly. The method however involves a lot of office computations in tracing the trips through a net work of stations. Unless there is a large net work of stations to take observations along the route of the vehicle, it is not easy to get the information of the routes followed by the vehicles.

Hence a large number of teams are required to take simultaneous observations when a large area is to be surveyed. However, this method is quite advantageous when the area under consideration is small, like a large intersection or a small business centre.

Return post card method

Pre-paid business reply post cards with return address are distributed to the road users at some selected points along the route or the cards are mailed to the owners of vehicles. The questionnaire to be filled in by the road user is printed on the card, along with a request for co-operation and purpose of the study. The distributing stations for the cards may be selected where vehicles have to stop as in case of a toll booth.

The method is suitable where the traffic is heavy. The personnel need not be skilled or trained just for distributing the cards. Only a part of the road users may return the cards promptly after filling in the desired details properly and correctly. If conclusions are drawn in such cases, it is likely that these may not give a true picture.

Tag on car method

In this method a pre-coded card is stuck on the vehicle as it enters the area under study. When the car leaves the cordon area the other observations are recorded on the tag. This method is useful where the traffic is heavy and moves continuously. But the method gives only information regarding the points of entry and exit and the time taken to traverse the area.

Home interview method

A random sample of 0.5 to 10 percent of the population it selected and the residences are visited by the trained personal who collect the travel data from each member of the house hold. Detailed information regarding the trips made by the members is obtained on the spot. The data collected may be useful either for planning the road net work and other roadway facilities for the vehicular traffic or for planning the mass transportation requirements of the passengers. The problem of stopping vehicle and consequent difficulties are avoided altogether. The present travel needs are clearly known and the analysis is also simple. Additional data including socio-economic and other details may be collected so as to be useful for forecasting traffic and transportation growth. But to have complete coverage of the entire cross section of the population is very tedious.

While planning for O and D studies at a place, it is necessary to decide the method of study. The selection of the method is dependent on the objective and the location. The influence of year and dates of study on the type and amount of traffic demand should be known. Care is needed in selecting the method of sampling and mode of data collection. The sample size should be decided keeping in view the desired accuracy and cost.

Work spot interview method

The transportation needs of work trips can be planned by collecting the O & D data at work spots like the offices, factories, educational institutions, etc. by personal interviews.

Presentation of O and D data

The data are presented in the following forms :

(i) Origin and destination tables are prepared showing number of trips between different zones.

(ii) *Desire lines* are plotted which is a graphical representation prepared in almost all O and D surveys. Desire lines are straight lines connecting the origin points with destinations, summarized into different area groups (See Fig. 5.7). The width of such desire lines is drawn proportional to the number of trips in both directions. The desire line density map easily enable to decide the actual desire of the road users and thus helps to find the necessity of a new road link, a diversion, a by-pass or a new bridge. These desire lines may be compared with the existing flow pattern

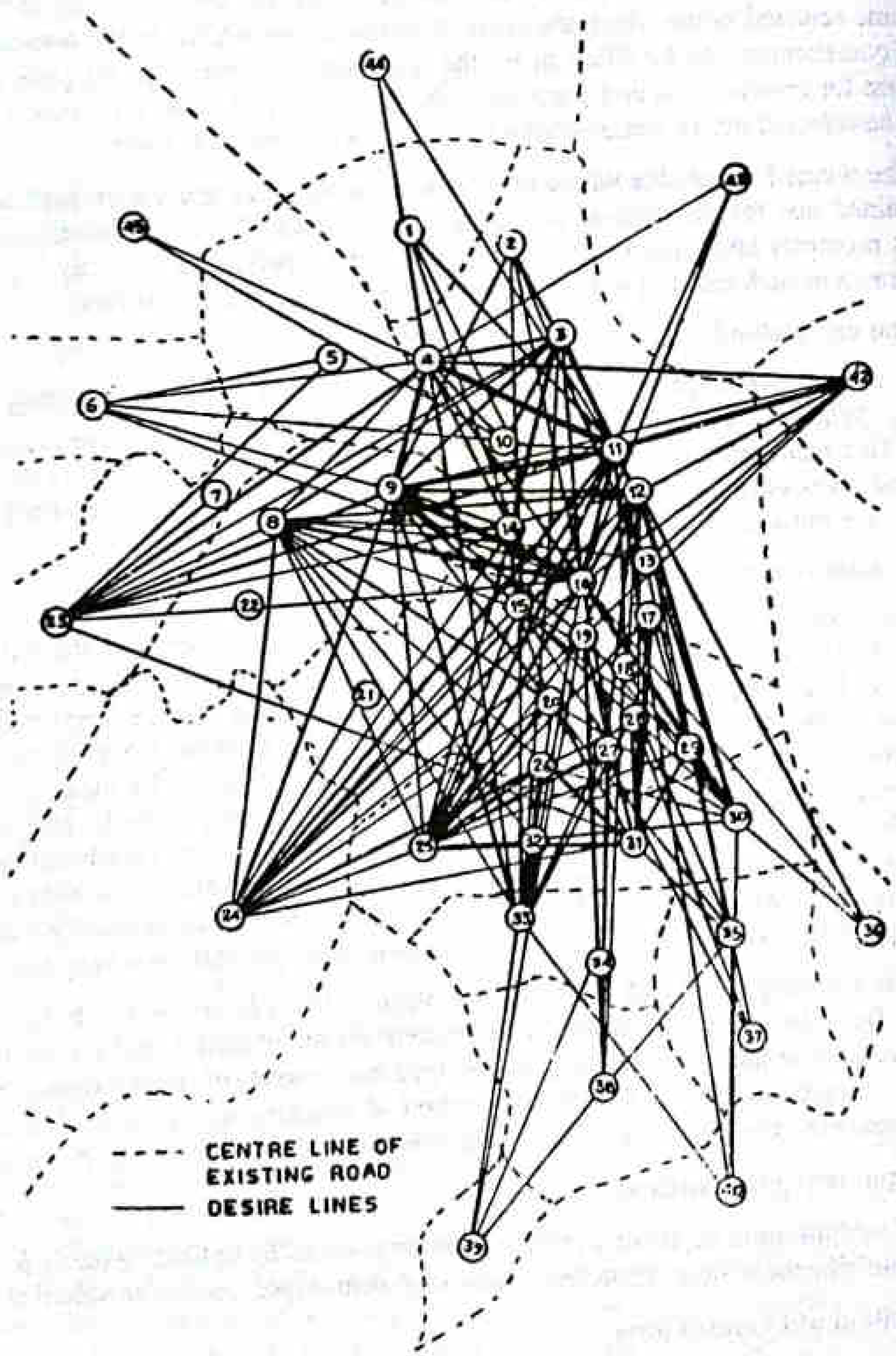


Fig. 5.7 Desire Lines

along the existing routes by superimposing one over the other with the help of tracing sheets.

(iii) The relative magnitude of the generated traffic and geometrical relationships of the zones involved may be represented by *pie charts*, in which circles are drawn, the diameter being proportional to the number of trips.

(iv) Contour lines may be plotted similar to topographic contours. The shape of the contours would indicate the general traffic need of the area.

Traffic flow characteristics and studies

Traffic stream generally has flow and counter flow along a common route, unless the stream is separated into pair of one-way flows by proper design or regulation.

The basic traffic manoeuvres are *diverging, merging and crossing* as shown in Fig. 5.8. Of all these, diverging on the left is the easiest movement causing least problem of the traffic conflicts. This is because the traffic is regulated on the left side. Merging from the left side also does not cause much of conflict. But diverging to the right and also merging from the right create conflicts and hazard to the traffic moving in the straight path. Transfer of a vehicle from one traffic lane to the next adjacent traffic lane is called *lane change* and this involves diverging and merging.

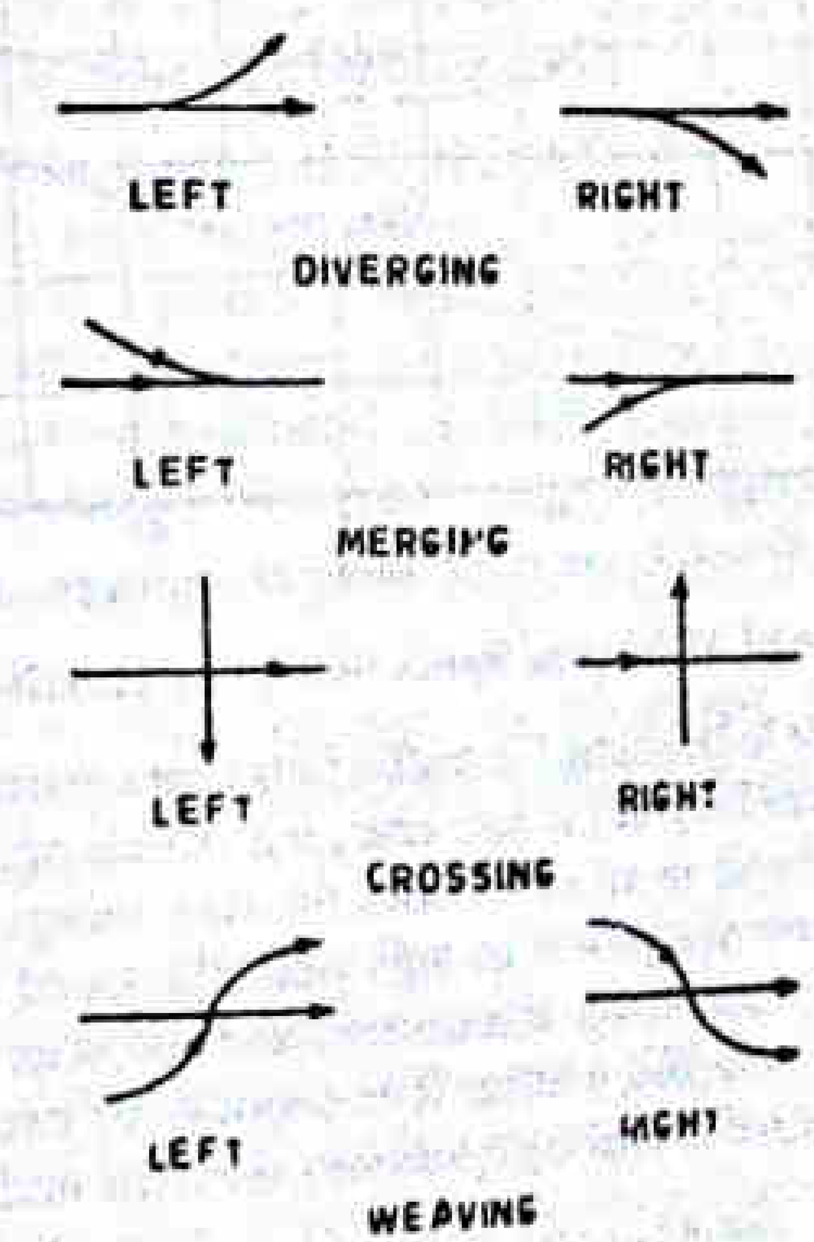


Fig. 5.8 Traffic Manoeuvres

The crossing traffic is the greatest problem in case of road intersections at level, because vehicles on one road have to stop before allowing the crossing stream of vehicles to cross their path. Thus the traffic capacity of two cross roads practically reduces to that of any one of the crossing roads or even lesser at the intersection. When a vehicle moves obliquely across the path of another vehicle moving in the same direction, at relatively small angle of crossing, the action is termed as *weaving*. The weaving manoeuvre may also be considered to consist of merging and diverging operations, along the stretch.

In two-way movements there may be crossing and over taking manoeuvres. The traffic stream characteristics are affected by the wide range of vehicles and road users, geometric feature of the road and intersections and other regulatory measures. Further the environmental conditions also affect the traffic stream flow.

The points to be particularly studied in traffic flow are the transverse and longitudinal distribution of vehicles on the various routes. The gaps ahead of each vehicle determine the longitudinal distribution of vehicles in one-way stream. See Fig. 5.9. The time interval between the passage of successive vehicles moving in the same lane and measured from head to head as they pass a point on the road is known as the *time headway*. The distance between successive vehicles moving in the same line measured from head at any instance is the *space headway* or the spacing of the vehicles in the stream. The variations in headway show the longitudinal distribution of the stream. The number of headways per unit time is dependent on the rate of traffic flow and is therefore a direct measure of traffic volume. With increase in speed of the traffic stream, the minimum space headway increases where as the minimum time headway first decreases and after reaching a minimum value at optimum speed on the stream, increases as shown in Fig. 5.9. Maximum flow or capacity flow is attained at this speed when the time headway is minimum.

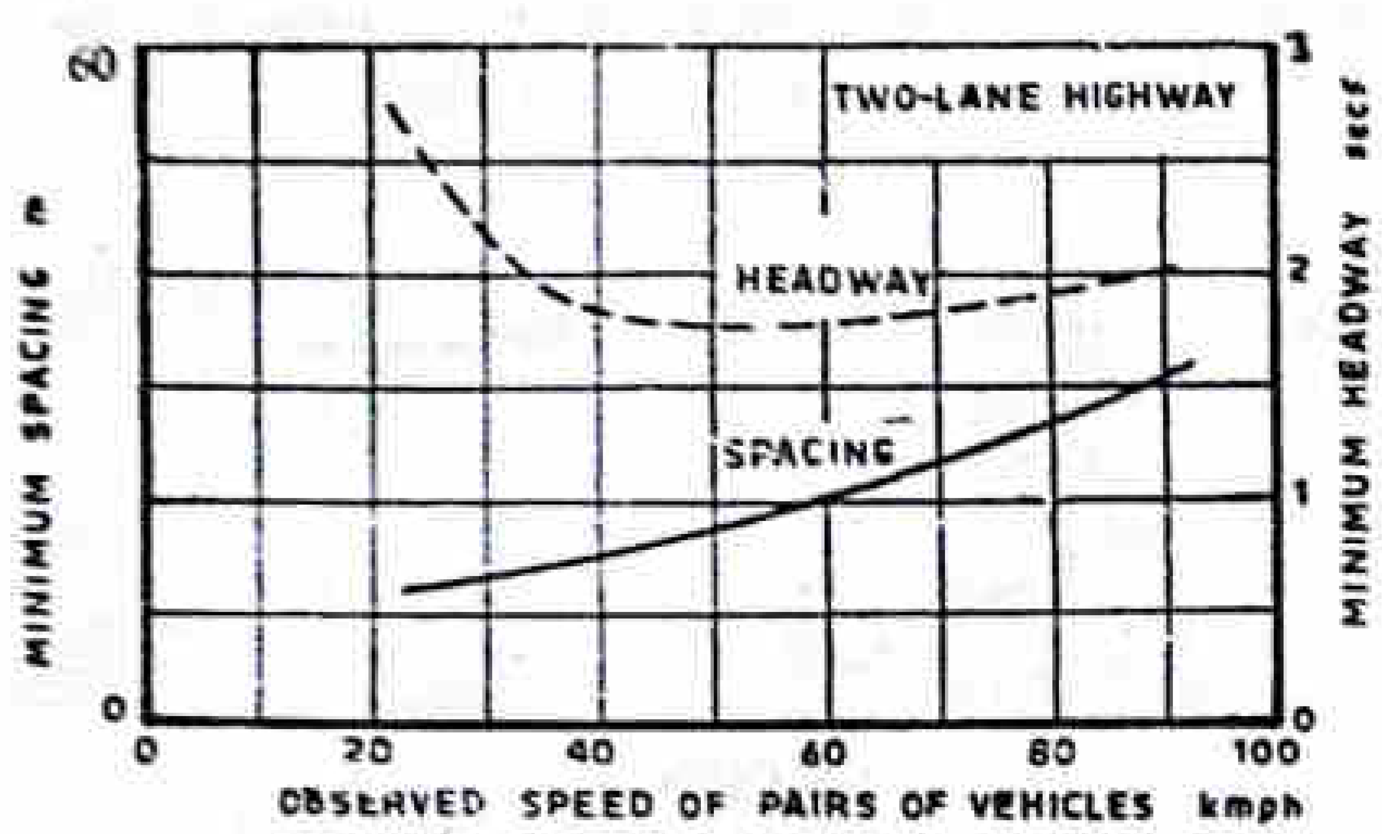


Fig. 5.9 Variation of Minimum Space and Time Headways with Speed

Another important factor to be studied in traffic flow characteristics is lane change in the traffic stream. When the headway of the lane changing vehicle rapidly decreases to almost zero, the lane change is forced; in all other cases the lane change may be optional. The frequency of demand for lane change will be high when the speed range of vehicles in the traffic stream is high. The lane change manoeuvres and characteristics would very much depend on the number of lanes and whether it is one-way or two-way movement. The merging, diverging, weaving and overtaking operations, all come under lane changes.

Study of traffic flow characteristics includes both transverse and longitudinal distribution of vehicles in the traffic stream and this is useful in geometric design features such as traffic capacity, volume, number of lanes and width of carriageway. The study is also very much needed to decide traffic regulatory measure like one-way movements and for the design of traffic control methods. Traffic flow study is particularly essential for large intersections.

Traffic capacity studies

Before studying details of traffic capacity, it may be worth while to define some of the related terms which are often used.

Traffic volume is the number of vehicles moving in a specified direction on a given lane or roadway that pass a given point or cross section during specified unit of time. Traffic volume is expressed as vehicles per hour or vehicles per day.

Traffic density is the number of vehicles occupying a unit length of lane of roadway at a given instant, usually expressed as vehicles per kilometre. Traffic volume is the product of the traffic density and traffic speed.

The highest traffic density will occur when the vehicles are practically at a stand still on a given route, and in this case traffic volume will approach zero.

Traffic capacity is the ability of a roadway to accommodate traffic volume. It is expressed as the maximum number of vehicle in a lane or a road that can pass a given point in unit time, usually an hour, i.e., vehicles per hour per lane or roadway. Capacity and volume are measures of traffic flow and have the same units. Volume represents an actual rate of flow and responds to variations in traffic demand, while capacity indicates a capability or maximum rate of flow with a certain level of service characteristics that can be carried by the roadway. The capacity of a roadway depends on a number of prevailing roadway and traffic conditions.

Basic capacity is the maximum number of passenger cars that can pass a given point on a lane or roadway during one hour under the most nearly ideal roadway and traffic conditions which can possibly be attained. Two roads having the same physical features will have the same basic capacity irrespective of traffic conditions, as they are assumed to be ideal. Thus basic capacity is the theoretical capacity.

Possible capacity is the maximum number of vehicles that can pass a given point on a lane or roadway during one hour under prevailing roadway and traffic conditions. The possible capacity of a road is generally much lower than the basic capacity as the prevailing roadway and traffic conditions are seldom ideal. In a worst case when the prevailing traffic condition is so bad that due to traffic congestion, the traffic may come to a stand still, the possible capacity of the road may approach zero.

When the prevailing roadway and traffic conditions approach the ideal conditions, the possible capacity would also approach the basic capacity. Thus the value of possible capacity varies from zero to basic capacity. For the purpose of design, neither basic capacity nor possible capacity can be adopted as they represent two extreme cases of roadway and traffic conditions.

Practical capacity is the maximum number of vehicle that can pass a given point on a lane or roadway during one hour, without traffic density being so great as to cause unreasonable delay, hazard or restriction to the driver's freedom to manoeuvre under the prevailing roadway and traffic conditions. It is the practical capacity which is of primary interest to the designers who strive to provide adequate highway facilities and hence this is also called design capacity.

Determination of theoretical maximum capacity

An estimate of theoretical maximum or basic capacity of a single lane may be made from the relation :

$$C = \frac{1000\,V}{S} \tag{5.3}$$

Here, C = capacity of a single lane, vehicle per hour

V = speed, kmph

S = average centre to centre spacing of vehicles, when they follow one behind the other as a queue or space headway, m

Thus the capacity depends upon the speed V and spacing S. The average spacing S between centre to centre of vehicles is equal to the average length of vehicle plus the clear spacing between the vehicles in the stream. The minimum clear spacing between vehicles are allowed for safe stopping of the rear vehicle in case the vehicle ahead suddenly stops. It is always found that drivers follow the vehicle ahead at a closer gap at a lower speeds and the clear spacing is increased instinctively at higher speeds of the traffic stream (Fig. 5.9).

Thus the space gap allowed by the driver of a followed vehicle depends on several factors such as

(i) speeds of leading and following vehicles

(ii) type and characteristics of the two vehicles

(iii) driver characteristics of the following vehicle

(iv) traffic volume to capacity ratio of the road section at the instant or the level of service

(v) The proportion of vehicle classes in the stream

(vi) road geometrics and

(vii) environmental factors

The assumption that space gap increases in direct proportion with the speed of the vehicle or that of the traffic stream is therefore a very much simplified one and gives only an approximate average value of the space gap between vehicles in the traffic stream. The space gap allowed by the following vehicle in a traffic stream is some time assumed to be equal to the distance travelled during the reaction time of the driver, assuming that the braking distances of the lead and the following vehicles are approximately equal. If the reaction time is t sec., the minimum space gap S_g is given by :

$$S_g = v\,t = 0.278\,V\,t,\ m$$

where v and V are average speeds in m/sec and kmph

The minimum space headway S in a traffic stream is therefore equal to the minimum space gap plus average length of vehicle L in the stream

$$S = S_g + L = 0.278\,V\,t + L$$

In a stream flow, as the driver of the following vehicle is quite alert, the average reaction time is found to be low; this value is often assumed to be 0.70 to 0.75 sec. In this analysis of overtaking sight distance (Art. 4.3.3) the value of reaction time has been assumed as 0.7 sec. in the empirical relation for spacing, i.e.,

$$S = (0.7\,v + L) = (0.2\,V + L),\ m$$

Thus a suitable value of S may be adopted in Eq. 5.3 to estimate the theoretical capacity of a traffic lane with homogeneous traffic flow.

It has been observed (as explained earlier) that with increase in speed of the traffic stream, the time headway decreases and after reaching a minimum value at an optimum speed, starts increasing (See Fig. 5.9). The maximum theoretical capacity of a traffic lane may therefore be obtained if the minimum time headway H_t is known.

$$C = \frac{3600}{H_t}$$

where C is the capacity, vehicles per hour (3600 second), and H_t is the minimum time headway in second.

The relationship between speed and maximum capacity of a traffic lane is shown in Fig. 5.10. The peak value of the theoretical maximum capacity if reached at an optimum speed. As the speed is increased further, the maximum capacity of the lane starts decreasing due to increase in headway at the speed range.

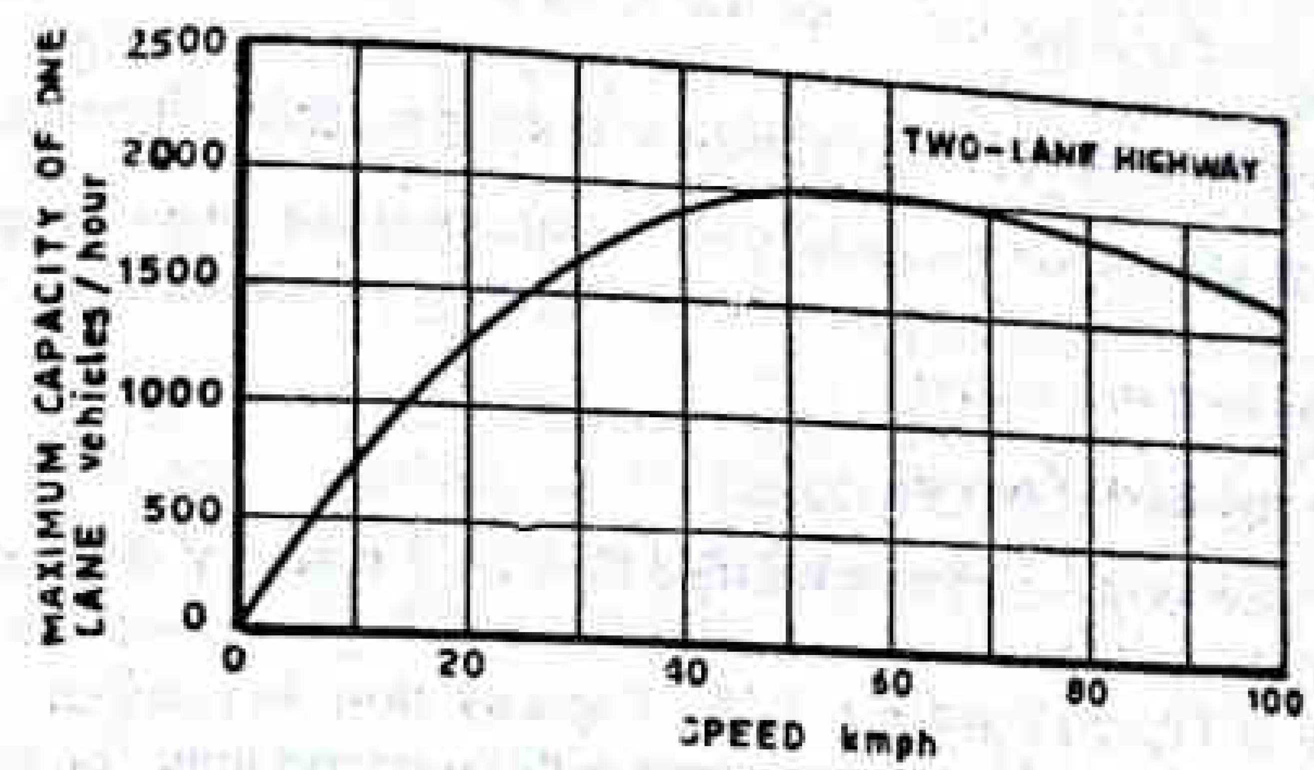


Fig. 5.10 Speed and Capacity

Factor affecting practical capacity

Some of the important factors that affect the practical capacity of a traffic lane are listed below :

(i) *Lane width* : As the lane width decreases, the capacity also decreases. The practical capacity of 3.0 m wide lane in a two-lane rural road may decrease to 76 percent of the capacity of a 3.5 m lane.

(ii) *Lateral clearance* : Vertical obstructions such as retaining walls, or parked vehicles near the traffic lane reduce the effective width of a lane and thus result in reduction in the capacity of lane. Further, restricted lateral clearance effects driving comfort and increases rates. A minimum clearance of 1.85 m from the pavement edge to the obstruction is considered desirable so that capacity is not affected adversely. When the distance from pavement edge to an obstruction decreases to 0.75 m on one side only, the capacity decreases to 96% and when this obstruction is on both sides, the percentage further decreases to 80% of the standard design capacity.

(iii) *Width of shoulders* : Narrow shoulders reduce the effective width of traffic lanes as the vehicles travel towards the centre of the pavement. When vehicles in emergency (like that of a tyre puncture or other break down) has to park on the shoulder of insufficient width, there is reduction in effective lane width resulting in a great reduction in the capacity of the lane.

(iv) *Commercial vehicles* : Large commercial vehicles like truck and buses occupy greater space and influence the other traffic in the same lane as well as the vehicles along the adjoining lanes. Also these heavy commercial vehicles may travel at lower speeds especially on grades.

(v) *Alignment* : If the alignment and geometrics are not upto the desired standards, the capacity will decrease. Particularly, restrictions to sight distance requirements cause reduction in capacity. Steep and long grades affect the capacity. When 60% of the road length has sub-standard OSD, the capacity decreases to 65% of the standard design capacity.

(vi) *Presence of intersections at grade* : Intersections restrict free flow of traffic and thus adversely affect the capacity. The capacity of an intersection of two roads crossing at grade will be slightly less than the road with lower capacity of the two. At signalized intersections as the vehicles have to stop alternately to allow crossing traffic, the capacity of the intersection will be further decreased. In order to provide consistent traffic flow and maximum capacity on important highways, is necessary to plan them as controlled access highways with grade separated intersections.

(vii) Other factors which affect the capacity are the stream speed, one or two way traffic movement, number of traffic lanes, vehicular and driver characteristics, composition of traffic and the traffic volume.

Design capacity and level of service

The capacity flow or the maximum possible flow on a roadway or a traffic lane is attained at particular optimum speed, the flow decreases at higher as well as lower speed values as shown in Fig. 5.11 and Fig. 5.18. Capacity flow is reached when all the vehicles flow as a stream at this optimum speed with no opportunity for overtaking; at this speed the level of service is considered to be fairly low when the volume of the road reaches the capacity or the volume to capacity ratio approaches a maximum possible value of 1.0.

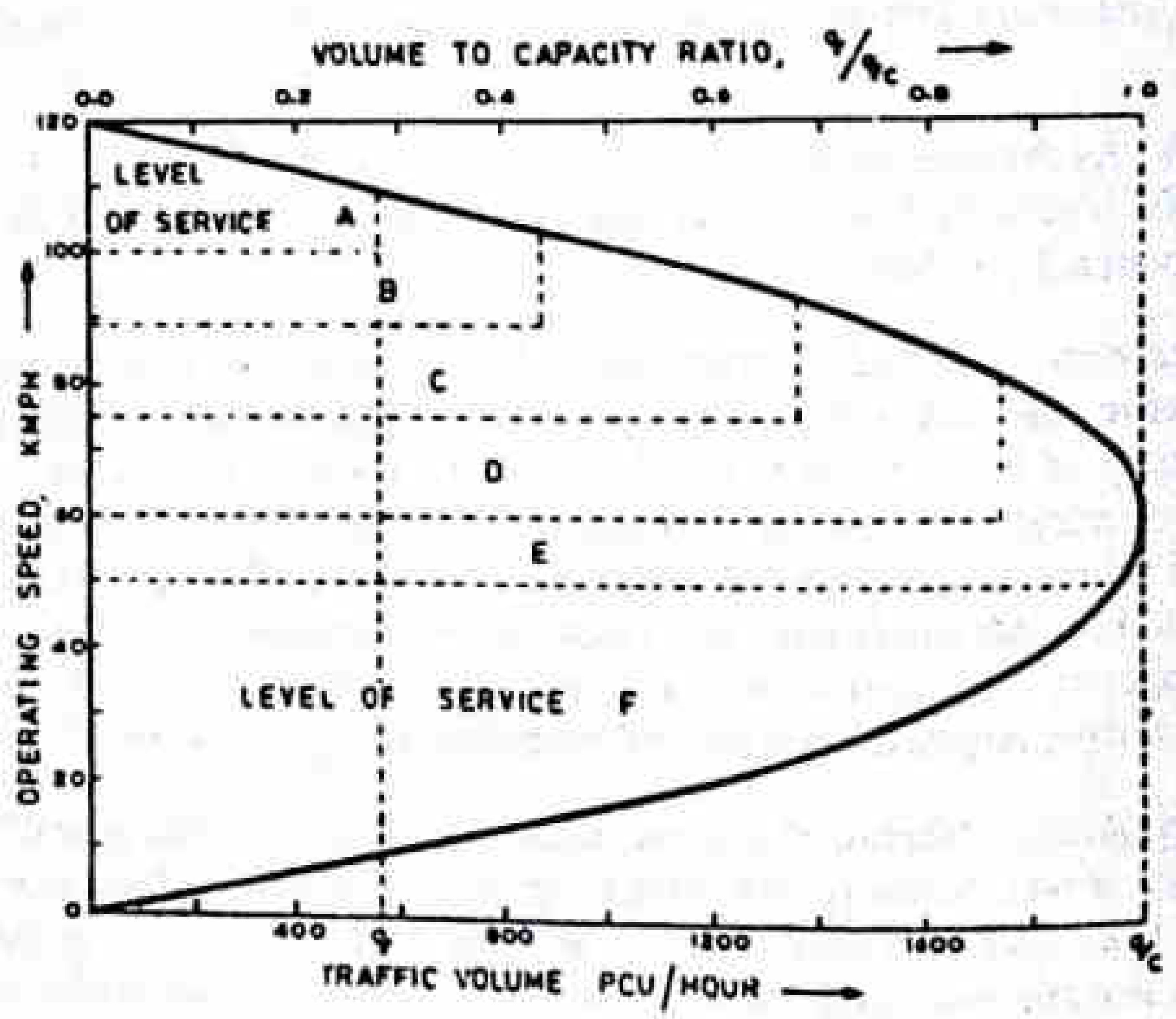


Fig. 5.11 General concept of level of service

Factors to be considered for the evaluation of level of service of a roadway in a comprehensive manner include the operating speed, travel time, traffic interruptions, freedom of manoeuvre, driving comfort, safety, economy etc. However, in order to simplify the level of service concept, two factors considered by the Highway Capacity Manual (HCM) are :

(i) the ratio of service volume to capacity q/q_c and

(ii) the operating or travel speed.

The HCM (Ref. 14) has suggested six levels of services A, B, C, D, E and F as shown in Fig. 5.11. Level of services A is considered to exist when the volume to capacity is so low that most of the individual vehicles have opportunities to travel at their oven desired speeds or free speeds and to overtake the slower vehicles at their will, this is possible at the highest speed range. With increase in the volume or the volume to capacity ratio, the operating speeds of faster vehicles and their opportunities to overtake decrease and the levels of service fall to decreasing values of B, C, D and E. Further, increase in vehicle arrivals causes further decrease in stream speed as well as in maximum flow, resulting in undue congestion and the lowest level of service F when forced flow conditions exist. The stream speed and the flow decrease to values much lower than the capacity flow condition and there will be practically no flow due to stopping of vehicles when the density attains the highest value or the 'jam density' as at a 'traffic jam'.

While designing the roadway facilities, care should be taken to adopt an appropriate value of design capacity, keeping in view the desired level of service. While planning and designing higher categories of highways, it is necessary to adopt design capacity values corresponding to high levels of service.

Passenger car unit (PCU)

Different classes of vehicles such as cars, vans, buses, trucks, auto rickshaw, motor cycles, pedal cycles, bullock carts, etc. are found to use the common roadway facilities without segregation on most of the roads in developing countries like India. The flow of traffic with unrestricted mixing of different vehicle classes on the roadways forms the heterogeneous traffic flow or the *mixed traffic flow*. The different vehicle classes have a wide range of static characteristics such as length, width etc. and dynamic characteristics such as speed, acceleration, etc. Apart from these, the driver behaviour of the different vehicle classes is also found to vary considerably. Therefore the mixed traffic flow characteristics are very much complex when compared to homogeneous traffic consisting of passenger cars only. It is rather difficult to estimate the traffic volume and capacity of roadway facilities under mixed traffic flow, unless the different vehicle classes are converted to one common standard vehicle unit. It is a common practice to consider the passenger car as the standard vehicle unit to convert the other vehicle classes and this unit is called Passenger Car Unit or PCU. Thus in mixed traffic flow, the traffic volume and capacity are generally expressed as PCU per hour or PCU/lane/hour and the traffic density as PCU per kilometre length of lane.

The PCU may be considered as a measure of the relative space requirement of a vehicle class compared to that of a passenger car under a specified set of roadway, traffic and other conditions. If the addition of one vehicle of a particular class in the traffic stream produces the same effect as that due to the addition of one passenger car, then that vehicle class is considered equivalent to the passenger car with a PCU value equal to 1.0. The PCU value of a vehicle class may be considered as the ratio of the capacity of a roadway when there are passenger cars only to the capacity of the same roadway when there are vehicles of that class only.

Factors affecting PCU values

The PCU values of different vehicle classes depend upon several factors. Some of these are listed below :

(i) Vehicles characteristics such as dimensions, power, speed, acceleration and braking characteristics.

(ii) Transverse and longitudinal gaps or clearances between moving vehicles which depends upon the speeds, driver characteristics and the vehicle classes at the adjoining spaces.

(iii) Traffic stream characteristics such as composition of different vehicle classes, mean speed and speed distribution of the mixed traffic stream, volume to capacity ratio etc.

(iv) Roadway characteristics such as road geometrics including gradient, curve, etc. access controls, rural or urban road, presence of intersections and the types of intersections.

(v) Regulation and control of traffic such as speed limit, one way traffic, presence of different traffic control devices, etc.

(vi) Environmental and climatic conditions.

Therefore the PCU value of a particular vehicle class may not remain a constant value as generally assumed. The important factors taken into account for a simple analysis of PCU values of different vehicle classes are :

(a) average speed of the vehicle class under the prevailing roadway and traffic conditions within the desired speed range.

(b) average length and width of the vehicle class.

(c) average transverse gap and longitudinal gap allowed between the vehicles of the same class in the speed range under consideration, during compact stream flow.

Based on the above factors, three different sets of PCU values have been worked out for :

(i) urban roads, mid block sections

(ii) signalized intersections and

(iii) kerb parking.

These are presented in Table 5.5 (See Ref. 24).

Table 5.5 Suggested PCU value for urban roads

S. No.	Vehicles class	PCU values of vehicle classes at : (i) Urban roads, mid-block sections	(ii) Signalised intersection	(iii) Kerb parking (parallel & angle)
1.	Car	1.0	1.0	1.0
2.	Bus and truck	2.2	2.8	3.4
3.	Auto rickshaw	0.5	0.4	0.4
4.	Two wheeler automobile	0.4	0.3	0.2
5.	Pedal cycle	0.7	0.4	0.1
6.	Bullock cart	4.6	3.2	1.2
7.	Hand cart	4.6	3.2	0.3

The Indian Roads Congress has given set of tentative PCU values or Equivalency Factors for rural road in even sections of plain terrain (Ref. 22) and these are presented in Table 5.6. However the IRC has suggested the set same of tentative Equivalency Factors for use on urban roads also (Ref. 23).

Table 5.6 Tentative Equivalency factors suggested by the IRC

S. No.	Vehicle class	Equivalency Factors
1.	Passenger car, tempo, autorickshaw, agricultural tractor	1.0
2.	Bus, truck, agricultural tractor-tailer unit	3.0
3.	Motor cycle, scooter and pedal cycle	0.5
4.	Cycle rickshaw	1.5
5.	Horse drawn vehicles	4.0
6.	Small bullock cart and hand cart	6.0
7.	Large bullock cart	8.0

Practical Capacity Values

The practical capacity values suggested by the IRC for the purpose of design of different types of roads in rural areas are given in Table 5.7.

Tentative capacity values of urban roads (mid-block sections, between intersections) suggested by the Indian Road Congress are given in Table 5.8.

Table 5.7 Capacity of different types of roads in rural areas

Types of road	Capacity PCU per day (both directions)
Single lane with 3.75 m wide carriageway and normal earthen shoulders	1000
Single lane roads with 3.75 m wide carriageway and 1.0 m wide hard shoulders	2500
Roads with intermediate lanes of width 5.5 m and normal earthen shoulders	5000
Two lane roads with 7.0 m wide carriageway and earthen shoulders.	10,000
Four lanes divided highway (depending on traffic, access control, etc.)	20,000 to 30,000

Table 5.8 Capacity of Urban Roads

No. of Traffic lanes and width	Traffic Flow	Capacity in PCU per hour for traffic condition: (i) Roads with no frontage access, no standing vehicles, very little cross traffic	(ii) Roads with frontage access, but no standing vehicle and high capacity intersections	(iii) Roads with free frontage access, parked vehicles & heavy cross traffic
Two lane (7.0 – 7.5)	One way	2400	1500	1200
Two lane (7.0 – 7.5)	Two way	1500	1200	750
Three lane (10.5 m)	One way	3600	2500	2000
Four lane (14.0 m)	One way	4800	3000	2400
Four lane (14.0 m)	Two way	4000	2500	2000
Six lane (21.0 m)	Two way	6000	4200	3600

Example 5.9

Estimate the theoretical capacity of a traffic lane with one way traffic flow at a stream speed of 40 kmph. Assume the average space gap between vehicles to follow the relation $S_g = 0.278\ Vt$ where V is the stream speed in kmph, t is the average reaction time = 0.7 sec; assume average length of vehicles = 5.0 m.

Solution

$$V = 40 \text{ kmph};\ t = 0.7 \text{ sec};\ L = 5.0 \text{ m}$$

$$S = 0.278\, V\, t + L = 0.278 \times 40 \times 0.7 + 5.0 = 12.78 \text{ m}$$

$$\text{Theoretical capacity } C = \frac{1000\,V}{S} = \frac{1000 \times 40}{12.78} = 3130 \text{ vehicles/hour/lane}$$

Parking Studies

The demand by automobile users of parking space is one of the major problems of highway transportation, especially in metropolitan cities. In industrial, commercial and residential places with multi-storeyed buildings, parking demand is particularly high. Parking studies are useful to evaluate the facilities available.

Various aspects to be investigated during parking studies are :

(i) *Parking demand*: The parking demand may be evaluate by different methods. One of the methods is by making cordon counts of the selected area and recording accumulation of vehicles during the peak hours by subtracting the outgoing traffic from the traffic volume entering the cordoned area.

One other method is by counting the number of vehicles parked in the area under study during different periods of the day; this method is useful when the parking demand is less than the space available for parking. By noting the registration number of each parked vehicle at any desired time interval (such as 30 minute, one hour, etc.) it is possible to estimate the duration of parking of each vehicle at the parking area. Another useful method of field study is by interviewing the drivers of parked vehicles, shop owners and other vehicle owners in the locality. This method is very useful when the parking demand in the study area is higher than the parking space available.

(ii) *Parking characteristics* : The study is directed to note the present parking practices prevalent in the area under consideration and the general problems in parking. In case of kerb parking, it is also necessary to study the parking pattern, interference to smooth flow of traffic and the accidents involved during parking and unparking operations.

(iii) *Parking space inventory* : The area under study is fully surveyed and a map is prepared showing all places where kerb parking and off-street parking facilities can be provided to meet the parking demands. The traffic engineer has to strike a balance between capacity and parking demands and to design proper facilities for parking. The design of parking facilities is presented in Art. 5.5.

Accident Studies

The problem of accident is very acute in highway transportation due to complex flow patterns of vehicular traffic presence of mixed traffic and pedestrians. Traffic accidents may involve property damages, personal injuries or even casualities. One of the main objectives of traffic engineering is to provide safe traffic movements. Road accident cannot be totally prevented, but by suitable traffic engineering and management measures, the accident rate can be decreased considerably. Therefore the traffic engineer has to carry out systematic accident studies to investigate the causes of accidents and to take preventive measures in terms of design and control. It is essential to analyse every individual accident and to maintain zone-wise accident records. The statistical analysis of accidents carried out periodically at critical locations or road stretches or zones will help to arrive at suitable measures to effectively decrease the accident rates.

The various objectives of the accident studies may be listed as :

(i) to study the causes of accidents and to suggest corrective treatment at potential location,

(ii) to evaluate existing designs,

(iii) to support proposed designs,

(iv) to carry out *before* and *after* studies and to demonstrate the improvement in the problem,

(v) to make computations of financial loss, and

(vi) to give economic justification for the improvements suggested by the traffic engineer.

Causes of accidents

There are four basic elements in a traffic accident :

(i) the road users

(ii) the vehicles

(iii) the road and its condition, and

(iv) environmental factors-traffic, weather etc.

The road user responsible for the accident may be the driver of one or more vehicles involved, pedestrians or the passengers. Vehicles involved in the accident may also be defective. The condition of the road surface or other existing geometric features or any of the environmental conditions of the road may not be upto the expectation causing an accident. To sum up, an accident may be caused due to a combination of several reasons and seldom due to one particular reason. Hence it is often not possible to pin point a particular single cause of an accident.

Various causes of accidents may hence be listed as given below :

(a) *Drivers* : Excessive speed and rash driving, carelessness, violation of rules and regulations, failure to see or understand the traffic situation, sign or signal, temporary effects due to fatigue, sleep or alcohol.

(b) *Pedestrians* : Violating regulations, carelessness in using the carriageway meant for vehicular traffic.

(c) *Passengers* : Alighting from or getting into moving vehicles.

(d) *Vehicle defects* : Failure of brakes, steering system, or lighting system, tyre burst and any other defect in the vehicles.

(e) *Road condition* : Slippery or skidding road surface, pot holes, ruts and other damaged conditions of the road surface.

(f) *Road design* : Defective geometric design like inadequate sight distance, inadequate width of shoulders, improper curve design, improper lighting and improper traffic control devices.

(g) *Weather* : Unfavourable weather condition like mist, fog, snow, dust, smoke or heavy rainfall which restrict normal visibility and render driving unsafe.

(h) *Animals* : Stray animals on the road

(i) *Other causes* : Incorrect signs or signals, gate of level crossing not closed when required, ribbon development, badly located advertisement boards or service stations etc.

Accident studies and records

The various steps involved in traffic accident studies are collection of accident data, preparation of reports, location file and diagrams, and application of the above records for suggesting preventive measures.

(i) Collection of accident data :

The collection of accident data is the first step in the accident study. Standard form for collecting the data are prepared, as suggested by the IRC (see Ref. 25).

The details to be collected are briefly mentioned here.

(a) *General* : Date, time, persons involved in the accident and their particulars, classification of accident like fatal, serious, minor etc.

(b) *Location* : Description and details of the location of accident.

(c) *Details of vehicles involved* : Registration number make and description of the vehicles, loading details, vehicular defects.

(d) *Nature of accident* : Condition of vehicles involved, details of collision, and pedestrians or objects involved, damages, injuries casuality etc.

(e) *Road and traffic conditions* : Details of road geometrics, whether the road is straight or curved, surface characteristics such as dry, wet or slippery etc. Traffic condition – type of traffic, traffic density, etc.

(f) *Primary causes of accident* : Various possible causes and the primary cause of the accident.

(g) *Accident costs* : The total cost of the accident computed in terms of rupees, of the various involvements like property damages, personal injuries and casualities.

(ii) Accident report :

The accident should be reported to police authorities who would take legal actions especially in more serious accidents involving injuries, casualities or severe damage to property. Accident report of the individuals involved may be separately taken. The accident data should be collected as given above and the accident report is prepared with all facts which might be useful in subsequent analysis, claims for compensation , etc.

(iii) Accident records :

The accident records are maintained giving all particulars of the accidents, location other details. The records may be maintained by means of location files, spot maps, collision diagrams and condition diagrams.

(a) *Location files* : These are useful to keep a check on the location of accident and to identify points of high accident incidence. Location fields should be maintained by each police station for the respective jurisdiction.

(b) *Spot maps* : Accident location spot maps show accidents by spots, pins or symbols on the map. A map of suitable scale say, 1 cm = 40 to 60 metre, may be used for spotting urban accidents. The common legend used for spot maps, are given in Fig. 5.12.

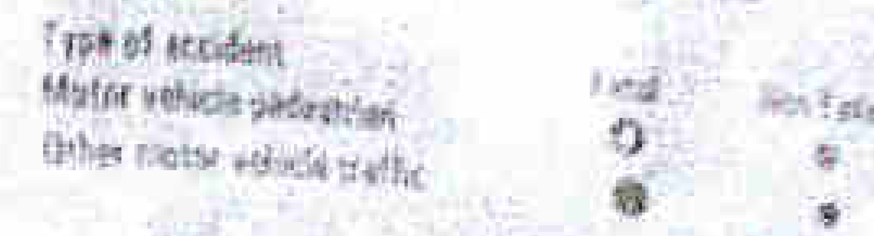


Fig. 5.12 Legend for Spot Maps

(c) *Condition diagram* : A condition diagram is a drawing to scale showing all important physical conditions of an accident location to be studied. The important features generally to be shown in this diagram with suitable dimensions marked there in an roadway limits, curves, kerb lines, bridges, culverts trees and all details of roadway conditions, obstruction to vision, property lines, signs, signals etc. There are standard symbols used in showing various details. The condition and collision diagrams may be combined together in a single sketch, if necessary.

(d) *Collision diagram* : These are diagrams showing the approximate path of vehicles and pedestrians involved in the accidents. Collision diagrams are most useful to compare the accident pattern before and after the remedial measures have been taken.

A typical collision diagram and symbols are shown in Fig. 5.13.

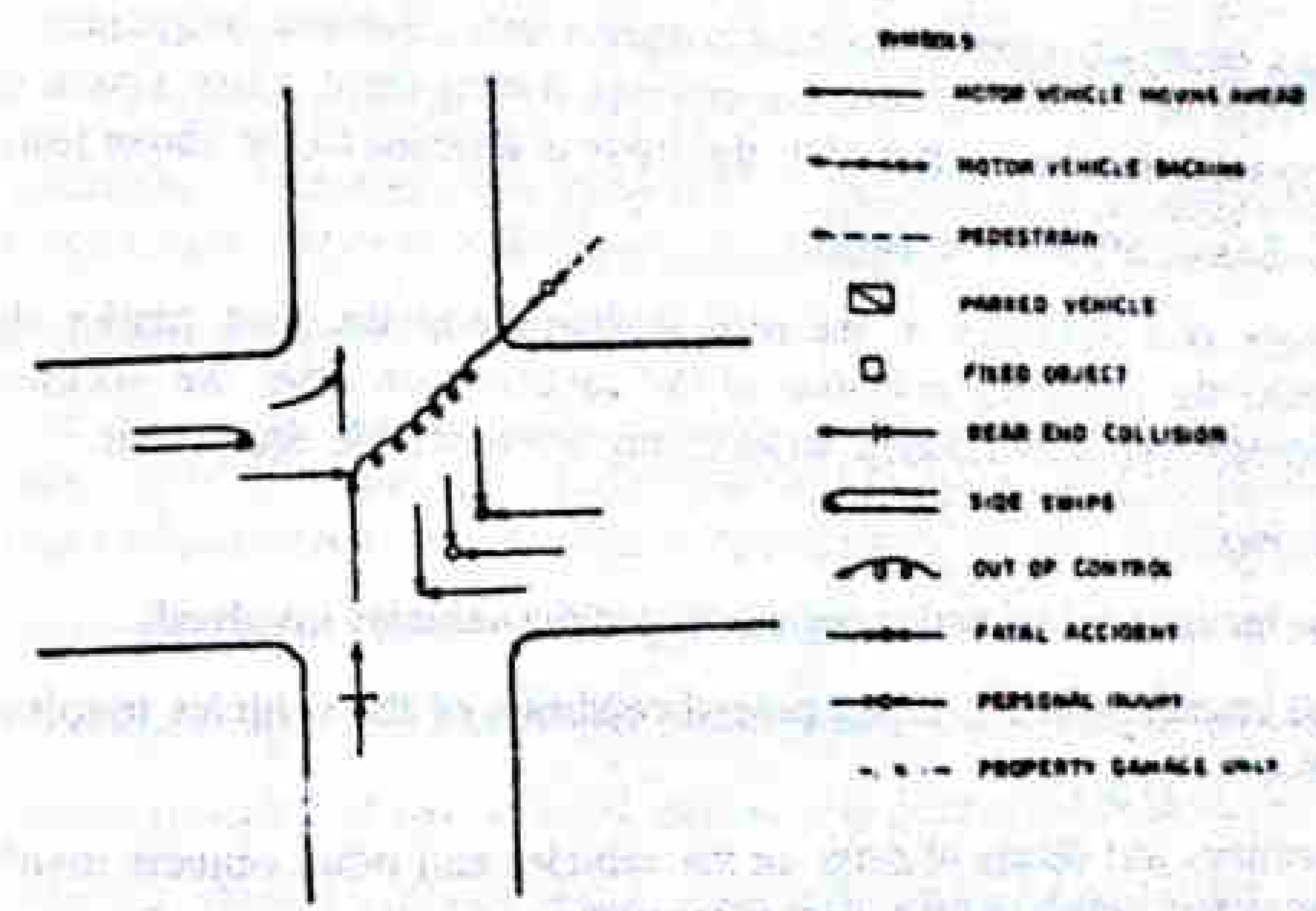


Fig. 5.13 Collision Diagram and Symbols

Accident investigations and studies therefore may be carried out scientifically in the following three stages :

Accident Investigations

Analysis of Individual Accidents

Statistical Analysis of Accidents

Accident investigations

The scientific approach for accident investigations suggested by the authors are summarized below. It is suggested that a mobile laboratory may be kept ready in every city. A bus equipped with essential instruments to measure the alcohol content in the breath, reaction time and other driver characteristics, skid resistance of pavement surface, etc, and a traffic engineer and his assistants may form the proposed mobile laboratory which should reach the accident spot as soon as possible after an accident. The following investigations may be carried out to enable analysis of the accident on a scientific basis.

(i) Recording General Observations

(a) Measurement of length of skid marks due to partial and full skidding.

(b) Recording the relative positions of vehicles and objects involved in the accident and collision diagram supplemented with photographs.

(c) Details of accident, injuries and damages

(d) Condition of pavement surface, shoulders and other surface through which the vehicles involved in the accident have moved; environmental conditions.

(e) Conditions diagram of accident locations with relevant measurements and dimensions.

(ii) Driver Tests

(a) Analysis of breath of drivers involved in the accident for alcohol content (using a suitable breathalyzer; if alcohol consumption is indicated above a prescribed limit, collection of blood sample for further analysis in a forensic laboratory). In case the driver is dead, tests may be conducted on the spinal fluid for estimating the alcohol content, if any,

(b) Tests on driver characteristics such as reaction time, distance judgement, angle of clear vision etc. If the accident has occurred during night, glare vision and glare recovery tests are to be conducted on the driver in addition to the above tests.

(iii) Skid Resistance of Pavement Surface

The average skid resistance of the road surface along the skid marks should be measured under the prevailing condition of the surface soon after the accident, using portable pendulum type skid resistance tester or any other suitable equipment.

(iv) Vehicle Tests

(a) Tests on the condition of brakes and steering of the vehicles involved.

(b) Tests on essential accessories and general condition of the vehicles involved in the accident.

(c) Characteristics and details of dents on the vehicles and other objects involved and the cross section details of the collapsed members.

(v) Probable Causes of the Accident

Assessment of the probable causes (primary, secondary and contributing causes) of the accident, its type, site conditions, position of the vehicles and other objects involved and other existing conditions.

(vi) Cost Analysis

Estimation of the cost of accident by working out the cost involved for the following items :

(a) Injuries and fatalities of persons involved

(b) Damages to the vehicles

(c) Property damages

(d) Other consequences including traffic delay

(e) Investigations and legal proceeding

Statistical analysis of accidents

The statistical analysis of road accidents help to assess the effectiveness of various measures to decrease the accident rate ; thus the analysis estimates the relative safety of road stretches or zones.

As the mobility increases the probability of accident also increases. The number of accidents are found to increase with the number of road users or the number of vehicles and pedestrians on the roads. As the vehicle movements and the population are on the increase, the total number of accidents in the study area are likely to increase year after year. The effectiveness of traffic engineering, enforcement and educational measures may therefore be judged from the changes in the annual accident rates, rather the total number.

The accident rate may either be expressed in terms of the number of vehicles and population or the vehicle movements (in vehicle-km) and the population. Relative accident rate may also be expressed in terms of various factors of the roadway and traffic. Accident-prone stretches of different roads may be assessed by finding the accident density per unit length of road. By statistical study of accident occurrence at a particular road or location or a zone of study for a long period of time, it is possible to predict with reasonable accuracy the probability of accident occurrence per day or the relative safety of different classes of road users in that location.

The reliability of the statistical analysis of accidents depends mainly on the reliability of the accident data, obtained from the accident records.

Analysis of individual traffic accidents

Each of the road accidents is analysed by the traffic engineer to draw sound conclusions. It is necessary to compute the original speeds of the vehicles involved in various types of accidents. Some of the typical modes of vehicular accidents are :

(i) A moving vehicle collides with a parked vehicle

(ii) Two vehicles approaching from different directions collide at an intersection

(iii) Head-on collision of two vehicles approaching from opposite directions

(iv) A moving vehicle collides with a stationary object like an electric pole, tree or a rigid structure.

The method of analysis for accident type (i) and (ii) mentioned above have been given below, as these are quite common type of accidents.

In order to simplify the analysis, some assumptions are made as discussed below :

(i) When skid marks are present, the skid distances are measured to find the actual braking distances and it is assumed that 100 percent skid has occurred. When skid marks are not at all visible, it may be assumed as a free collision, without the brakes being applied.

(ii) When two vehicles of masses m_a and m_b with speeds v_a and v_b collide, if it is assumed that both are perfectly plastic bodies, both would move together with the same speed v′ after impact and the relationship is given by :

$$(m_a v_a + m_b v_b) = (m_a + m_b) v'$$

If both the bodies are perfectly elastic, the coefficient of restitution will be unity and relationship is given by :

$$(v_a - v_b) = (v_b' - v_a')$$

In case the coefficient of restitution e is known, then the relation is :

$$e\,(v_a - v_b) = (v_b' - v_a')$$

The actual values of the coefficient of restitution should be either known or suitably assumed.

(iii) The impact of the vehicles may be either direct or oblique, at a known angle.

(iv) The friction coefficient of the pavement surface under the prevailing conditions may either be determined from field test or be suitably assumed. However the friction coefficient is assumed to be uniform throughout the skid.

Analysis of speed from skid distance

The basic equation for finding the braking distance or skid distances S for a vehicle of weight W to slow down from speed v_1 to v_2 m/sec is obtained by equating the reduction in kinetic energy with the work done against the frictional force i.e.,

$$\frac{W}{2g}\left(v_1^2 - v_2^2\right) = W.f.S$$

where f is the average friction factor or skid resistance developed.

$$S = \frac{v_1^2 - v_2^2}{2gf} \qquad (5.4)$$

Substituting the values of g = 9.8 m/sec^2 and the speed in V_1 and V_2 kmph,

$$S = \frac{V_1^2 - V_2^2}{254\,f} \qquad (5.5)$$

If the skid distance S is measured from the skid marks, the initial speed v_1 may be calculated from the relation

$$v_1 = \sqrt{v_2^2 + 2g\,f\,S} \qquad (5.6)$$

In kmph units,

$$V_1 = \sqrt{v_2^2 + 254\,f\,S} \qquad (5.7)$$

If the vehicle comes to a stop after the skid distance S, then v_2 would be zero in this equation.

Case (i)

Collision of moving vehicle with parked vehicle

Suppose a vehicle A, moving with speed v_1 m/sec skids through a distance S_1 after the application of the brakes, collides with a parked vehicle B and the two vehicles skid together through a distance S_2 before coming to a stop. The problem is to estimate the initial speed of vehicle A, v_1 m/sec or V_1 kmph.

(a) *Before collision* the vehicle A of weight W_A moving with initial speed v_1 before applies brakes, skids through a distance S_1 and attains a speed v_2 m/sec just before collision. From Equation 5.6 the relationship between v_1 and v_2 is given by :

$$v_1^2 = v_2^2 + 2g\,f\,S_1 \qquad (5.8a)$$

(b) *At collision* with stationary vehicle B of weight W_B, both start moving together at speed v_3 m/sec. Here a perfectly plastic impact is assumed with e = 0. The relation between v_2 and v_3 is given by the momentum equation

$$\frac{W_A}{g}.v_2 = \frac{W_A + W_B}{g}.v_3 \text{ or } v_2 = \frac{W_A + W_B}{W_A}.v_3$$

Substituting in Eq. 5.8a

$$v_1^2 = \left(\frac{W_A + W_B}{W_A}\right)^2 v_3^2 + 2g\,f\,S_1 \qquad (5.8b)$$

(c) After collision vehicles A and B skid through distance S_2 before coming to a stop (velocity v_3 reducing to v_4 = 0). The relation of v_3 in terms of S_2 is obtained again from Eq. 5.9 :

$$v_3^2 = 2g\,f\,S_2$$

Substituting v_3 in Eq. 5.8b,

$$v_1^2 = \left(\frac{W_A + W_B}{W_A}\right)^2 2g\,f\,S_2 + 2g\,f\,S_1$$

$$v_1 = \sqrt{\left(\frac{W_A + W_B}{W_A}\right)^2 2g\,f\,S_2 + 2g\,f\,S_1} \qquad (5.9)$$

In kmph units,

$$V_1 = \sqrt{254\,f\left[S_2\left(\frac{W_A + W_B}{W_A}\right)^2 + S_1\right]} \qquad (5.10)$$

where W_A = weight of moving vehicle, kg

W_B = weight of parked vehicle, kg

f = average friction coefficient

S_1 = initial skid distance before collision, metre

S_2 = skid distance of both the vehicles together after collision, metre.

When the vehicle A does not apply brakes and does not skid before collision,

S_1 = 0 in Equation 5.9 and 5.10

Example 5.10

A vehicle of weight 2.0 tonne skids through a distance equal to 40 m before colliding with another parked vehicle of weight 1.0 tonne. After collision both the vehicles skid through a distance equal to 12 m before stopping.

Compute the initial speed of the moving vehicle. Assume coefficient of friction as 0.5.

Solution

Method (i) By Steps :

This problem may be solved easily in three steps without using the Eq. 5.9 or 5.10.

Let the original speed of the vehicle be v_1 m/sec., reduced to v_2 m/sec by applying brakes and skidding through s_1 = 40 m; just after the collision, let both vehicles A and B start moving together with speed v_3 m/sec and finally stop, $v_4 = 0$, after skidding through distance s_2 = 12 m, f = 0.5.

(a) After collision

Loss in kinetic energy of both vehicles together = work done against frictional force

i.e., $$\frac{(W_a + W_b)}{2g}(v_3^2 - v_4^2) = (W_a + W_b)\, f\, s_2$$

$$\therefore \frac{v_3^2}{2g} = 0.5 \times 12$$

or $$v_3 = \sqrt{2 \times 9.8 \times 0.5 \times 12} = \sqrt{117.6} \text{ m/sec}$$

(b) At collision

Momentum before impact = momentum after impact

i.e., $$\frac{W_a . v_2}{g} = \frac{(W_a + W_b)\, v_3}{g}$$

$$\frac{W_a + W_b}{W_a} = \frac{2+1}{2} = \frac{3}{2}$$

$$v_2 = \frac{(W_a + W_b)}{W_a} v_3 = \frac{3}{2}\sqrt{117.6} \text{ m/sec}$$

(c) Before collision

Loss of kinetic energy = work done against braking force in reducing the speed

$$\frac{W_a}{2g}(v_1^2 - v_2^2) = W_a \times f \times s_1$$

$$v_1^2 = 2g\, f\, s_1 + v_2^2 = 2 \times 9.8 \times 0.5 \times 40 + \frac{9}{4} \times 117.6$$

$$= 656.6$$

$$v_1 = 25.6 \text{ m/sec}$$

Original speed $V_1 = 3.6 \times 25.6 = 92.2$ kmph

Method (ii) By using Equation 5.10

$$V_1 = \sqrt{254 f \left[s_2 \left(\frac{W_a + W_b}{W_a} \right)^2 + s_1 \right]}$$

$$= \sqrt{254 \times 0.5 \left[12 \left(\frac{3}{2} \right)^2 + 40 \right]} = 92.2 \text{ kmph}$$

Case (ii)

Two vehicles approaching from right angles collide

Two vehicles A and B on approaching an intersection are assumed to skid on application of brakes; they collide with each other and skid further in different directions as illustrated in Fig. 5.14 (a), (b) and (c). The direction of the skidding vehicles after collision in this case depends on the initial speeds of the two vehicles and their weights.

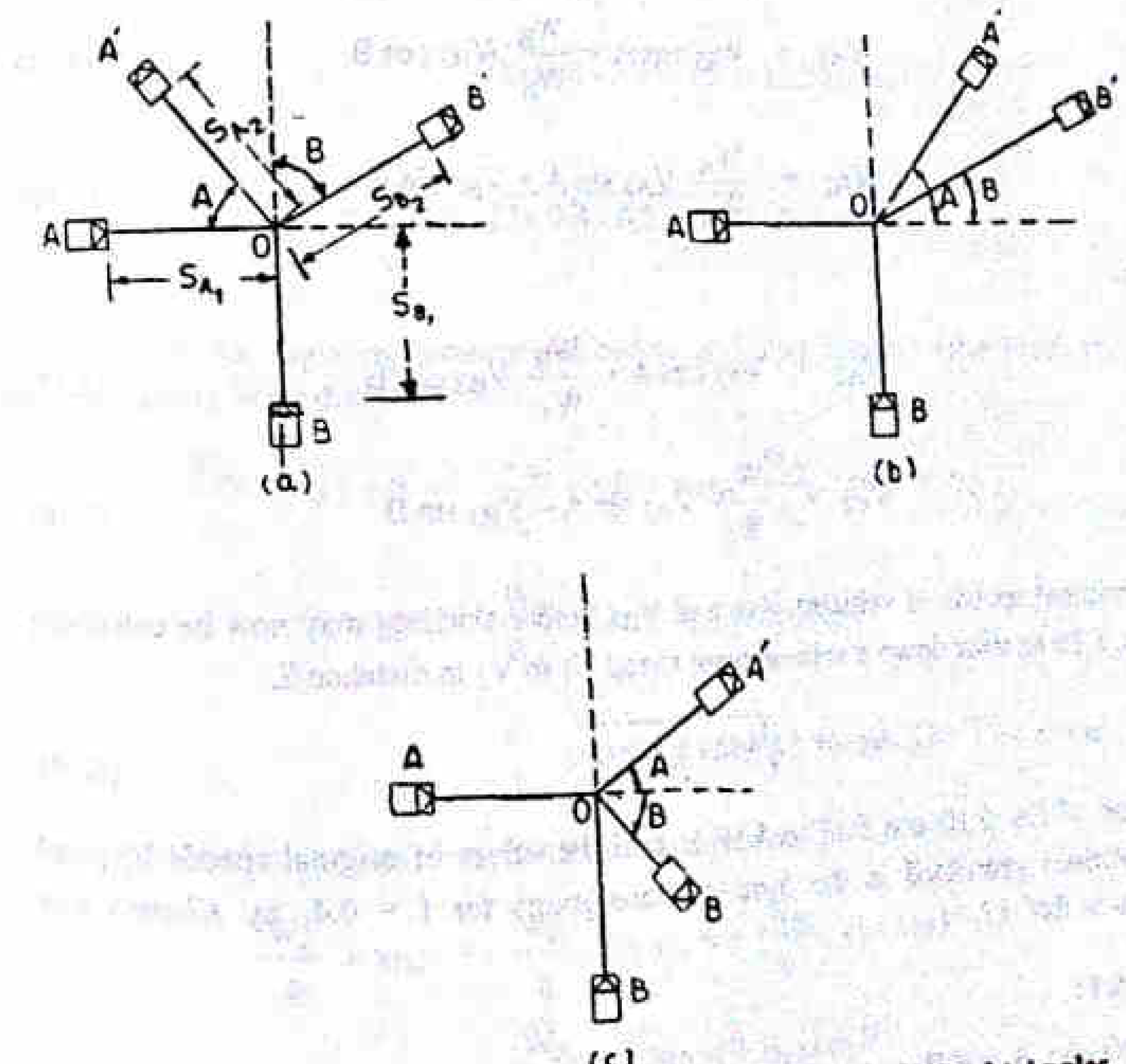


Fig. 5.14 Collision of Two Vehicles approaching from Right Angles

If S_{A2} and S_{B2} are the skid distance of the vehicles after the collision (Fig. 5.14) the speeds of vehicles V_{A3} and V_{B3} just after collision may be found from relations :

$$V_{A3} = \sqrt{254 f S_{A2}} \quad (5.11)$$

$$V_{B3} = \sqrt{254 f S_{B2}} \quad (5.12)$$

The momentum of the vehicles just after collision may be found using the speed values from Eq. 5.11 and 5.12 and these resolved in the original direction of motion of the two vehicles. As per the assumption, the momentum before collision is taken equal to the momentum after collision. For the three cases of right angle collision shown in Fig. 5.14, the speeds of the vehicles just before collision, but after skidding through distances S_{A1} and S_{B1} i.e. V_{A2} or V_{B2} are obtained by the relations given below for the three cases :

Case (a)

$$V_{A2} = \frac{W_{B2}}{W_A} V_{B3} \sin B - V_{A3} \cos A \quad (5.13)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A + V_{B3} \cos B \quad (5.14)$$

Case (b)

$$V_{A2} = V_{A3} \cos A + \frac{W_B}{W_A} V_{B3} \cos B \quad (5.15)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A + V_{B3} \sin B \quad (5.16)$$

Case (c)

$$V_{A2} = V_{A3} \cos A + \frac{W_B}{W_A} V_{B3} \cos B \quad (5.17)$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A - V_{B3} \sin B \quad (5.18)$$

The original speeds of vehicles V_{A1} and V_{B1} before skidding may now be calculated using Eq. 5.10 to slow down a vehicle from speed V_1 to V_2 in distance S.

i.e.,

$$V_1 = \sqrt{254 f S_1 + V_2^2} \quad (5.19)$$

Solution of Eq. 5.10 and 5.13 to 5.19 to find the values of original speeds V_{A1} and V_{B1} have been presented in the form of nomograms for f = 0.4, by *Khanna and co-authors* in Ref. 17.

Example 5.11

Two vehicles A and B approaching at right angles, A from West and B from South, collide with each other. After the collision, vehicle A skids in a direction 50° North of West and vehicle B, 60° East of North. The initial skid distances of the vehicles A and B are 38 and 20 m respectively before collision. The skid distances after collision are 15 and 36 m respectively. If the weights of vehicles B and A are 6.0 and 4.4 tonnes, calculate the original speeds of the vehicles. The average skid resistance of the pavement is found to be 0.55.

Solution

Method (i) By Steps

Let the initial speeds of vehicles A and B before brake application be v_{A1} and v_{B1}, the speeds just before collision, after skidding through S_{A1} = 38 and S_{B1} = 20 m be v_{A2} and v_{B2}, the speeds just after collision, be v_{A3} and v_{B3} and the final speed when the vehicles come to a stop is zero; after skidding through further distance S_{A2} = 15 and S_{B2} = 36 m; f = 0.55 (Refer Fig. 5.14 a).

(a) After collision

Loss in kinetic energy of each vehicle = work done against skid resistance

For vehicle A,
$$\frac{W_A v_{A3}^2}{2g} = W_A f. S_{A2}$$

$$\therefore v_{A3} = \sqrt{2g f S_{A2}}$$

$$= \sqrt{2 \times 9.8 \times 0.55 \times 15} = 12.7 \text{ m/sec}$$

Similarly,

$$V_{B3} = \sqrt{2 \times 9.8 \times 0.55 \times 36} = 19.7 \text{ m/sec}$$

(b) At collision

From Fig. 5.14a, equating momentums before and after impacts after resolving the momentums along West-East direction,

$$\frac{W_A}{g} \times v_{A2} + 0 = \frac{W_B}{g} \sin B \times v_{B3} - \frac{W_A}{g} . \cos A \times v_{A3}$$

$$\therefore v_{A2} = \frac{W_B}{W_A} . \sin B \, v_{B3} - v_{A3} \cos A$$

$$= \frac{6}{4} . \sin 60 \times 19.7 - 12.7 \times \cos 50 = 17.4 \text{ m/sec}$$

Resolving the momentums along South-North direction,

$$\frac{W_B}{g} \times v_{B2} = \frac{W_A}{g} v_{A3} \sin A + \frac{W_B}{g} . V_{B3} \cos B$$

$$\therefore v_{B2} = \frac{W_A}{W_B} v_{A3} \sin A + v_{B3} \cos B$$

$$= \frac{4}{6} \times 12.7 \times \sin 50 + 19.7 \cos 60 = 16.4 \text{ m/sec}$$

(c) After collision

Loss in kinetic energy due to brake application = work done against brake application

i.e., $\frac{W_A}{2g}(v_{A1}^2 - v_{A2}^2) = W_A . f . S_{A1}$

$$\therefore v_{A1}^2 = 2g\,f\,S_{A1} + v_{A2}^2$$

$$= 2 \times 9.8 \times 0.55 \times 38 + 17.4^2 = 712.4$$

$$v_{A1} = 26.7 \text{ m/sec}, V_{A1} = 26.7 \times 3.6 = 96 \text{ kmph}$$

Similarly:

$$V_{B1}^2 = 2g\,f\,S_{B1} + v_{B2}^2 = 2 \times 9.8 \times 0.55 \times 20 + 16.4^2$$

$$= 484.6$$

$$v_{B1} = 22 \text{ m/sec}, V_{B1} = 22 \times 3.6 = 79.2 \text{ kmph}$$

Method (ii) By using the equations

Using Eq. 5.11 and 5.12, speeds of vehicles just after collision,

$$V_{A3} = \sqrt{254\,f\,S_{A2}} = \sqrt{254 \times 0.55 \times 15} = 45.8 \text{ kmph}$$

$$V_{B3} = \sqrt{254 \times 0.55 \times 36} = 70.9 \text{ kmph}$$

Using Eq. 5.13 and 5.14, speeds of vehicles just before collision,

$$V_{A2} = \frac{W_B}{W_A} V_{B3} \sin B - V_{A3} \cos A$$

$$= \frac{6}{4} \times 70.9 \times \sin 60 - 45.8 \cos 50 = 92.1 - 29.4$$

$$= 62.7 \text{ kmph}$$

$$V_{B2} = \frac{W_A}{W_B} V_{A3} \sin A + V_{B3} \cos B$$

$$= \frac{4}{6} \times 45.8 \sin 50 + 70.9 \cos 60$$

$$= 23.4 + 35.5 = 58.9 \text{ kmph}$$

Original speeds of vehicles before application of brakes are obtained using Eq. 5.19.

$$V_{A1} = \sqrt{254\,f\,S_{A1} + V_{A2}^2}$$

$$= \sqrt{254 \times 0.55 \times 38 + 62.7^2} = 96 \text{ kmph}$$

$$V_{B1} = \sqrt{254\,f\,S_{B1} + V_{B2}^2}$$

$$= \sqrt{254 \times 0.55 \times 20 + 58.9^2} = 79 \text{ kmph}$$

Thus by both the methods, the original speeds of vehicles A and B before the application of brakes are 96 and 79 kmph respectively.

Measures for the reduction in accident rates

The various measures to decrease the accident rates may be divided into three groups :

(i) Engineering

(ii) Enforcement

(iii) Education

These three measures are generally termed "3-Es". The details of these measures are given below.

Engineering Measures

(a) Road design : The geometric design features of the road such as sight distances, width of pavement, horizontal and vertical alignment design details and intersection design elements are checked and corrected if necessary. The pavement surface characteristics including the skid resistance values are checked and suitable maintenance steps taken to bring them upto the design standards. Where necessary by-passes may be constructed to separate through traffic from local traffic. To minimise delay and conflicts at the intersections, it may be essential to design and construct grade separated intersections or fly overs.

(b) Preventive maintenance of vehicles : The braking system, steering and lighting arrangements of vehicles plying on the roads may be checked at suitable intervals and heavy penalties levied on defective vehicles. These measures are particularly necessary for public carriers.

(c) Before and after studies : The record of accidents and their patterns for different locations are maintained by means of collision and condition diagrams. After making the necessary improvements in design and enforcing regulation, it is again necessary to collect and maintain the record of accidents "before and after" the introduction of preventive measures to study their efficiency. A typical example of before and after study at an intersection is shown in Fig. 5.15.

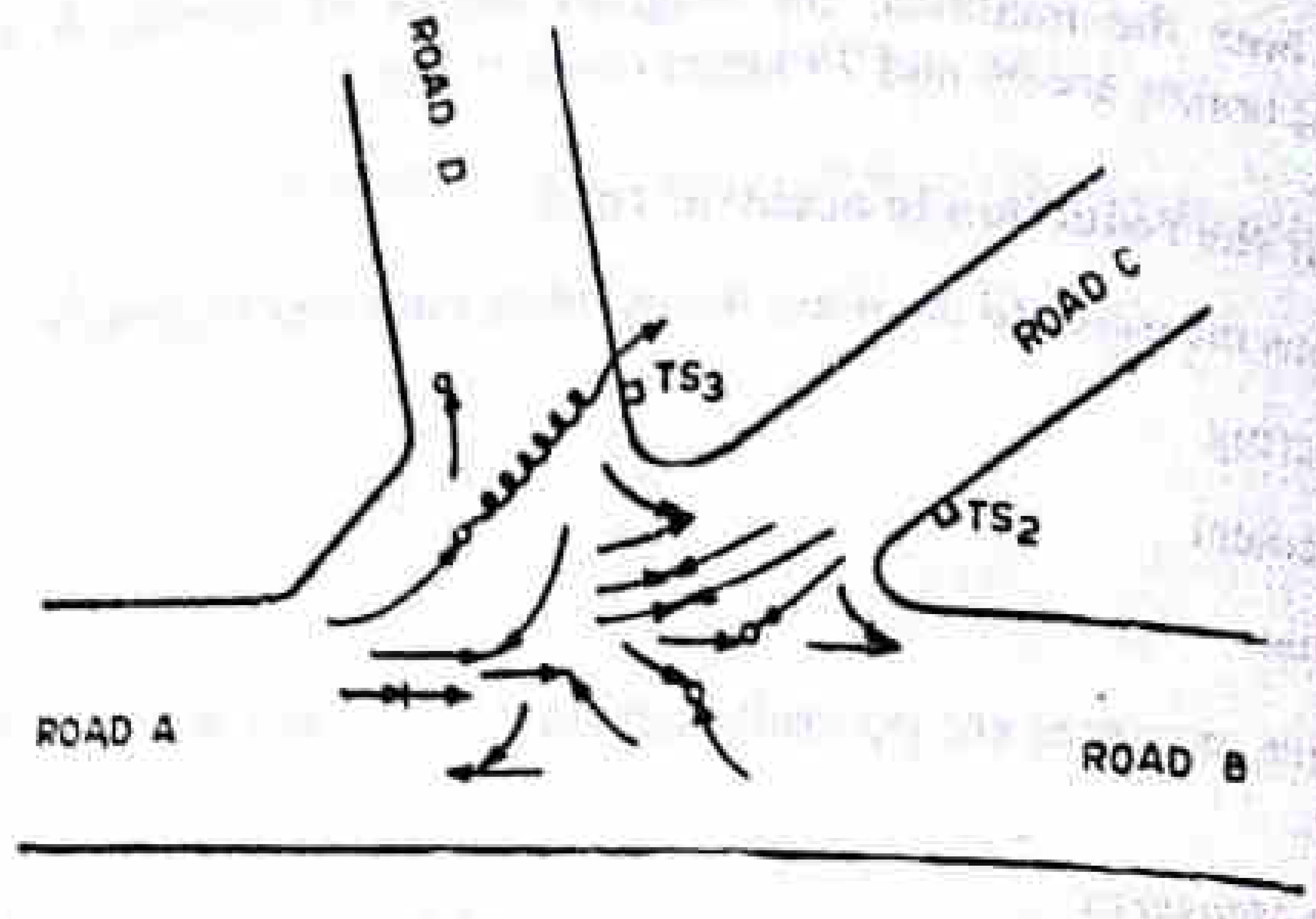
(d) Road lighting : Proper road lighting can decrease the rate of accidents during night, due to poor visibility. Lighting is particularly desirable at intersections, bridge sites and at places where there are restrictions to traffic movements.

Enforcement Measures

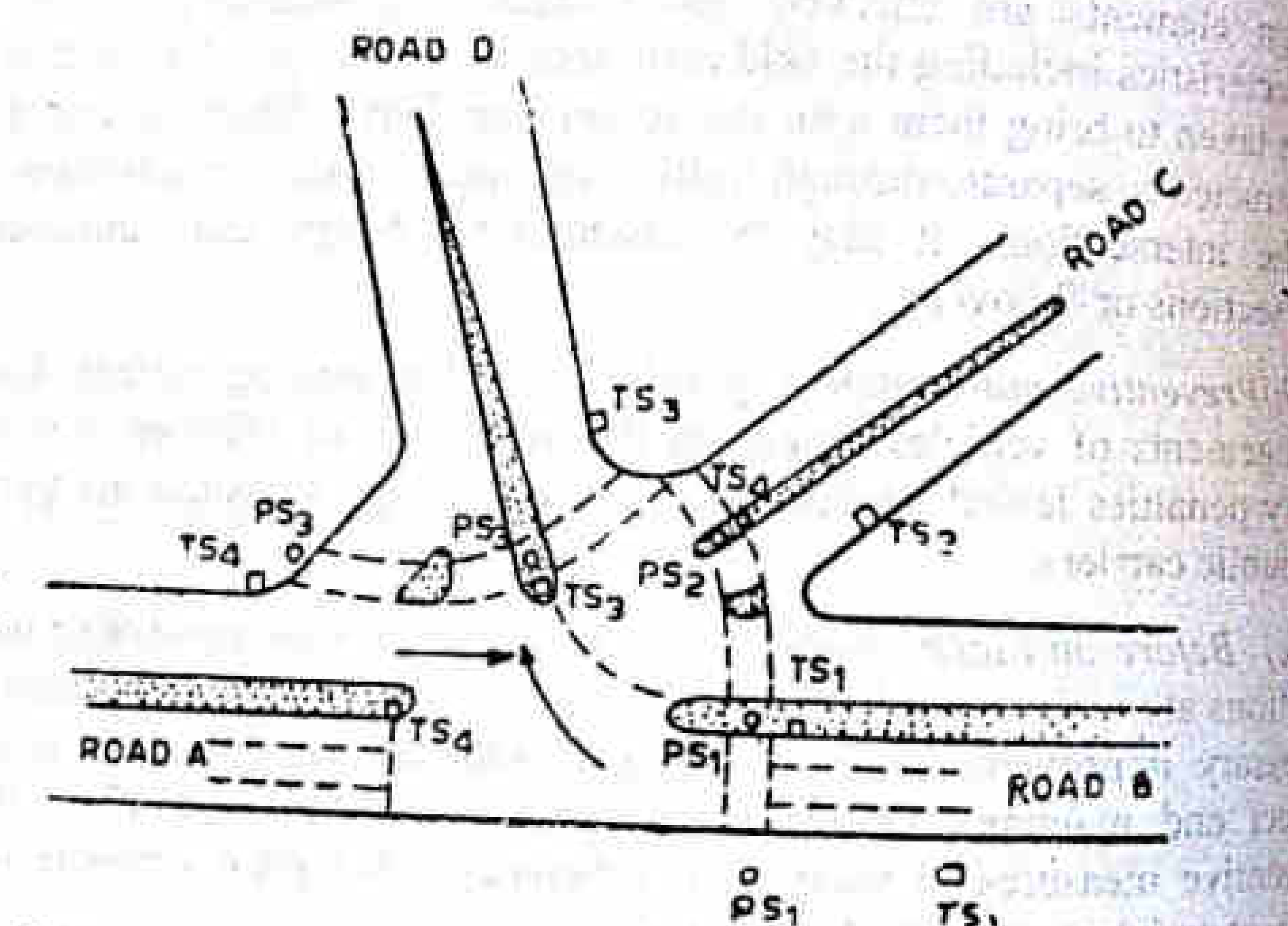
The various measures of enforcement that may be useful to prevent accidents at spots prone to accidents are enumerated here. The motor vehicle rules are revised from time to time to make them more comprehensive.

(a) *Speed control* : To enable drivers of buses to develop correct speed habits tachometers may be fitted so as to give the record of speeds. Also surprise checks on spot speed of all fast moving vehicles should be done at selected locations and timings and legal actions on those who violate the speed limits should be taken.

(b) Traffic control devices : Signals may be re-designed or signal system be introduced if necessary. Similarly proper traffic control device like signs, markings or channelizing islands may be installed wherever found necessary.



Uncontrolled movements of vehicles and pedestrians
Accidents : Twelve in a period of two months



(i) Additional traffic signals (T.S.) installed and signal timings re-designed
(ii) Pedestrian signals (P.S.) installed and pavement markings made for pedestrian crossings and control over other vehicular manoeuvers.
(iii) Divisional islands and channelizing islands provided by widening roads A-B.
Accidents : only on in a period of two months

Fig. 5.15 Typical Case of Before and After Study

(c) Training and supervision : The transport authorities should be strict in testing and issuing license to drivers of public service vehicles and taxis. Even the drivers who have passed the requisite tests should be kept under proper supervision and be trained in proper defensive driving. Driving license of the driver may be renewed after specified period, only after conducting some tests to check whether the driver is fit.

(d) Medical check : The drivers should be tested for vision and reaction time at prescribed intervals, say, once in three years.

(e) Special precautions for commercial vehicles : It may be insisted on having a conductor or attendant to help and give proper direction to drivers of heavy commercial vehicles.

(f) Observance of law and regulation : This is one of the most essential steps in enforcement for prevention of accidents. Traffic or transport authorities should send study groups of trained personnel, assisted by police to different locations to check whether the traffic regulations are being followed by the road users and also to enforce the essential regulations. The study group can provide useful data for deciding about the necessity of revision of certain traffic regulations.

Educational Measures

(a) Education of road users : It is very essential to educate the road users for the various precautionary measures to use the road way facilities with safety. The passengers and pedestrians should be taught the rules of the road, correct manner of crossing etc. This may be possible by introducing necessary instruction in the schools for the children. Posters exhibiting the serious results due to carelessness of road users may also be useful. The Indian Roads Congress has recently prepared Highway Safety Code and the document on Road Safety for school children and an Instruction Manual on Road Safety Education is under preparation.

(b) Safety drive : Imposing traffic safety week when the road users are properly directed by the help of traffic police and transport staff is a common means of training the public these days. Roads users should be impressed on what should and what should not be done, with the help of films and documentaries. Training courses may be conducted for drivers. The IRC has been orgainsing Highway Safety Workshop in different regions of the country.

5.2.4 Relationship Between Speed, Travel Time, Volume, Density and Capacity

In the operation and planning of traffic facilities the relationship between the fundamental stream flow variables is important.

The *travel time* per unit length of road is inversely proportional to the speed. If T is travel time and V is the speed (kmph).

$$T \text{ (min/km)} = \frac{60}{V} \qquad (5.20a)$$

or

$$T \text{ (sec/km)} = \frac{3600}{V} \qquad (5.20b)$$

Figure 5.16 shows the relationship between travel time and speed. It is seen that at higher speeds, the rate of saving in travel time decreases.

The fundamental relationship between traffic volume, density and speed may be given by the general equation of traffic flow :

$$q = K\,V_s \qquad (5.21)$$

where

q = the average volume of vehicles passing a point during a specified period of time; (vehicles per hour)

K = the average density or number of vehicles occupying a unit length of roadway at a given instant (vehicles/km)

V_s = space-mean speed of vehicles in a unit roadway length (kmph)

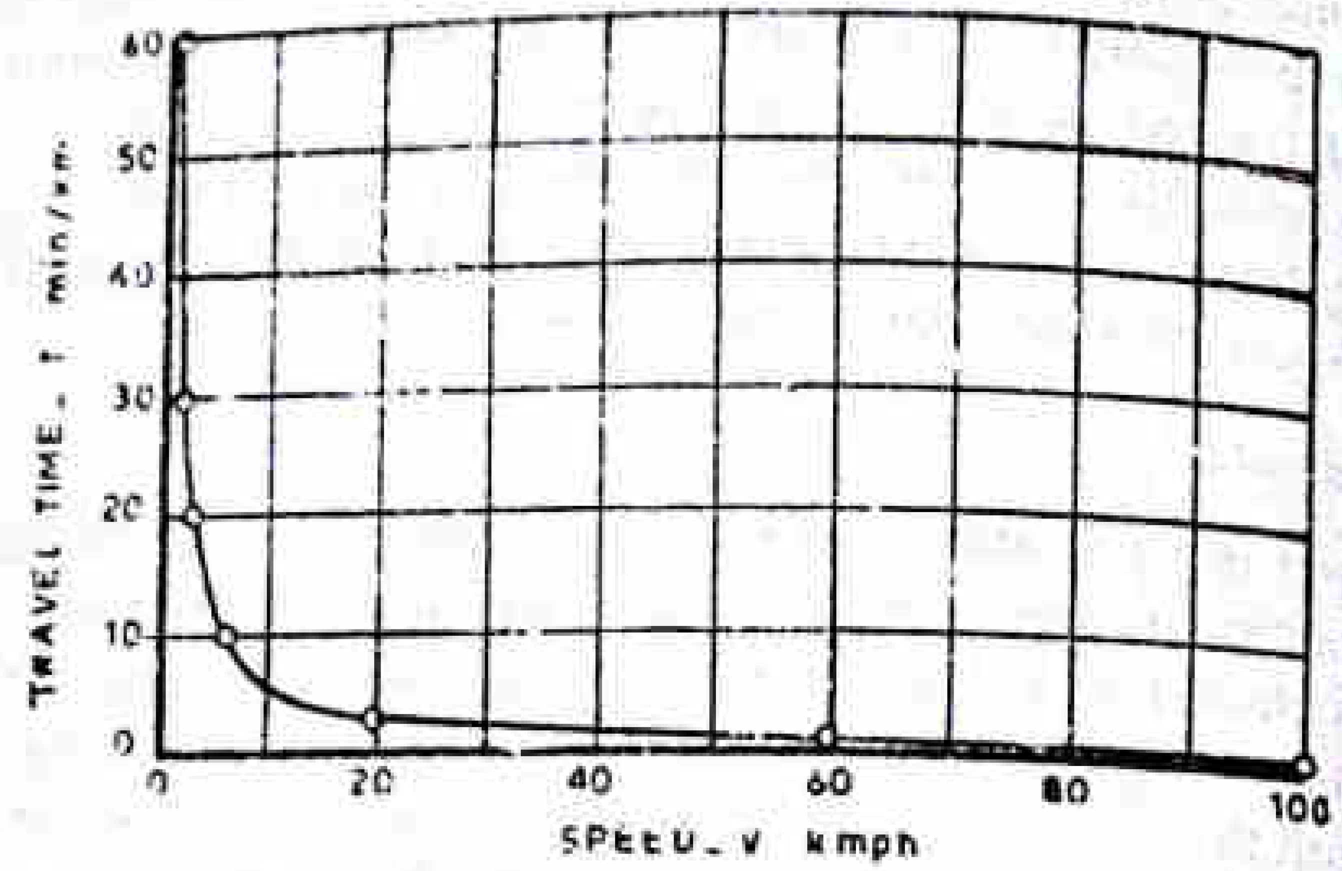


Fig. 5.16 Travel Time and Speed

With increase in speed of vehicles in a unit roadway length the average density decreases. This is because the spacing between the vehicles increases with increase in speed, as discussed in Art. 5.2.3. Field observations of speed and density made simultaneously, have indicated that approximately straight line relationship between speed and density could be obtained for a good range of speeds, particularly when the speed is not high. As the speed approaches zero i.e., towards stand-still maximum density is obtained. Figure 5.17 illustrates a hypothetical case based on the simple model of straight line relationship between speed and density. The dashed line shows the actual trend of observations at higher speeds and the extension by the dotted lines is the hypothetical case.

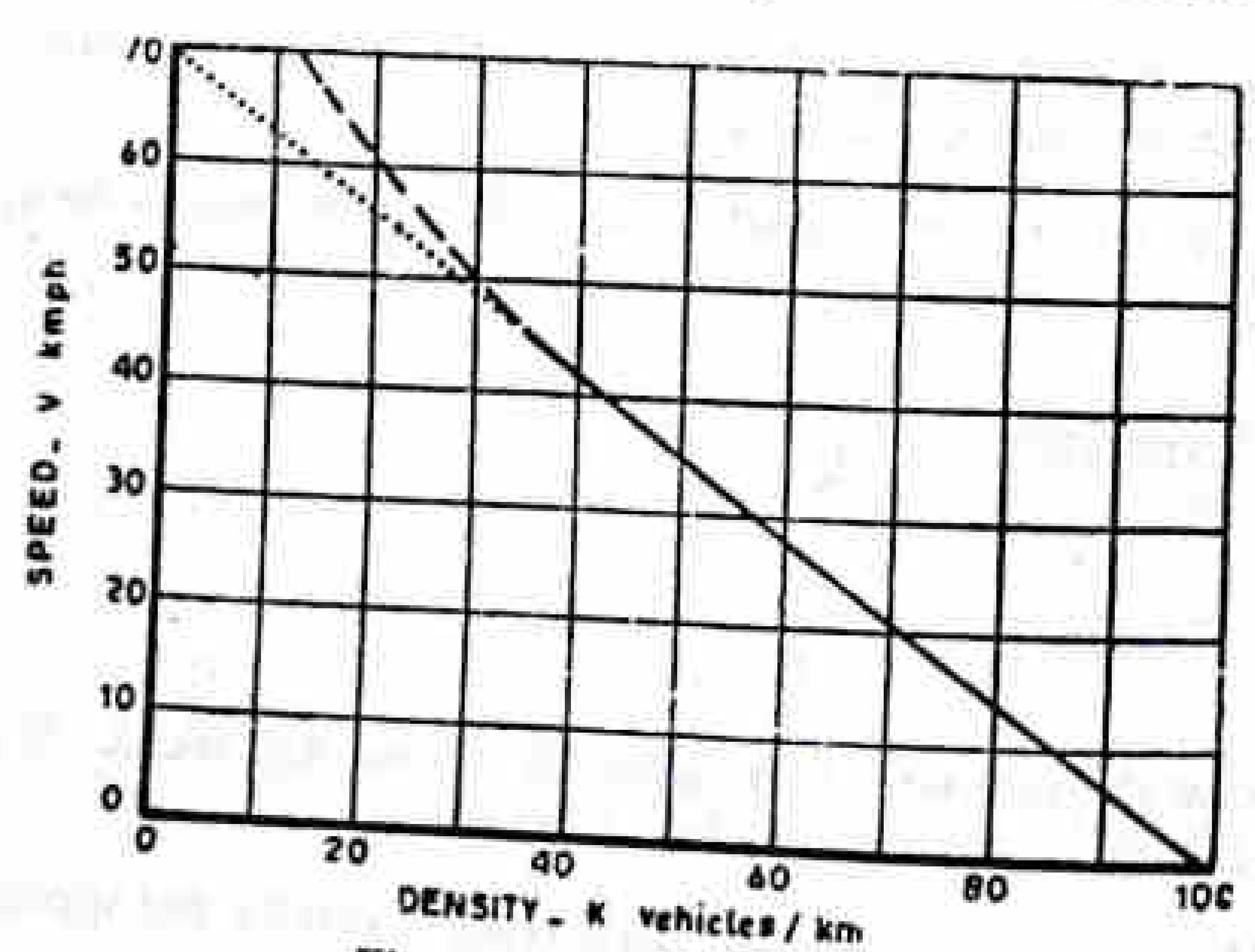


Fig. 5.17 Speed and Density

It is difficult to measure density directly, in practice. Hence the relationship between volume, density and speed (Eq. 5.5) is often used. The value of density K may be obtained by rewriting Eq. 5.5.

$$K \text{ (vehicles/km)} = \frac{q \text{ (vehicles/hr)}}{v \text{ (km/hr)}} \qquad (5.22)$$

It is obvious that at very low speeds, the traffic volume would also be low; zero speed indicates zero flow or volume. With increasing speeds, traffic volume also increases upto

a certain limit as the time head-way H_t initially decreases. But as the speed further increases, the spacing between the vehicles becomes so large that the time headway between the vehicles also increases and thus the volume decreases. The relationship between speed and volume of traffic for a hypothetical case is shown in Fig. 5.18. It may be seen that with each value of volume there are two speed values. The dashed line indicates the trend based on observation. Extension by the dotted line is based on the simplified hypothesis.

When the speed of the traffic flow decreases and becomes zero, the density attains the maximum value whereas, volume becomes zero. For increasing values of speeds, density decreases, whereas the volume increases upto a certain limit (Fig. 5.17 and 5.18). At high speeds, the volume starts decreasing and density keeps on further reducing. Eventually if a hypothetical case is considered when volume approaches zero at very high speeds, the density also approaches zero as shown in Fig. 5.19. Thus there is a maximum flow in road corresponding to some optimum values of speed and density.

In Eq. 5.21 if any two of the three stream variables i.e., volume, speed and density, are known, the third may be determine. The traffic volume and speed may be measured easily in the field, whereas it is difficult to measure the density. Hence by measuring the values of traffic volume and speed, it is possible to compute the density. When the three relations given in Fig. 5.17, 5.18 and 5.19 are combined and plotted on three mutually perpendicular axes, the surface obtained may be visualized as the basic traffic stream equation as illustrated in Fig. 5.20.

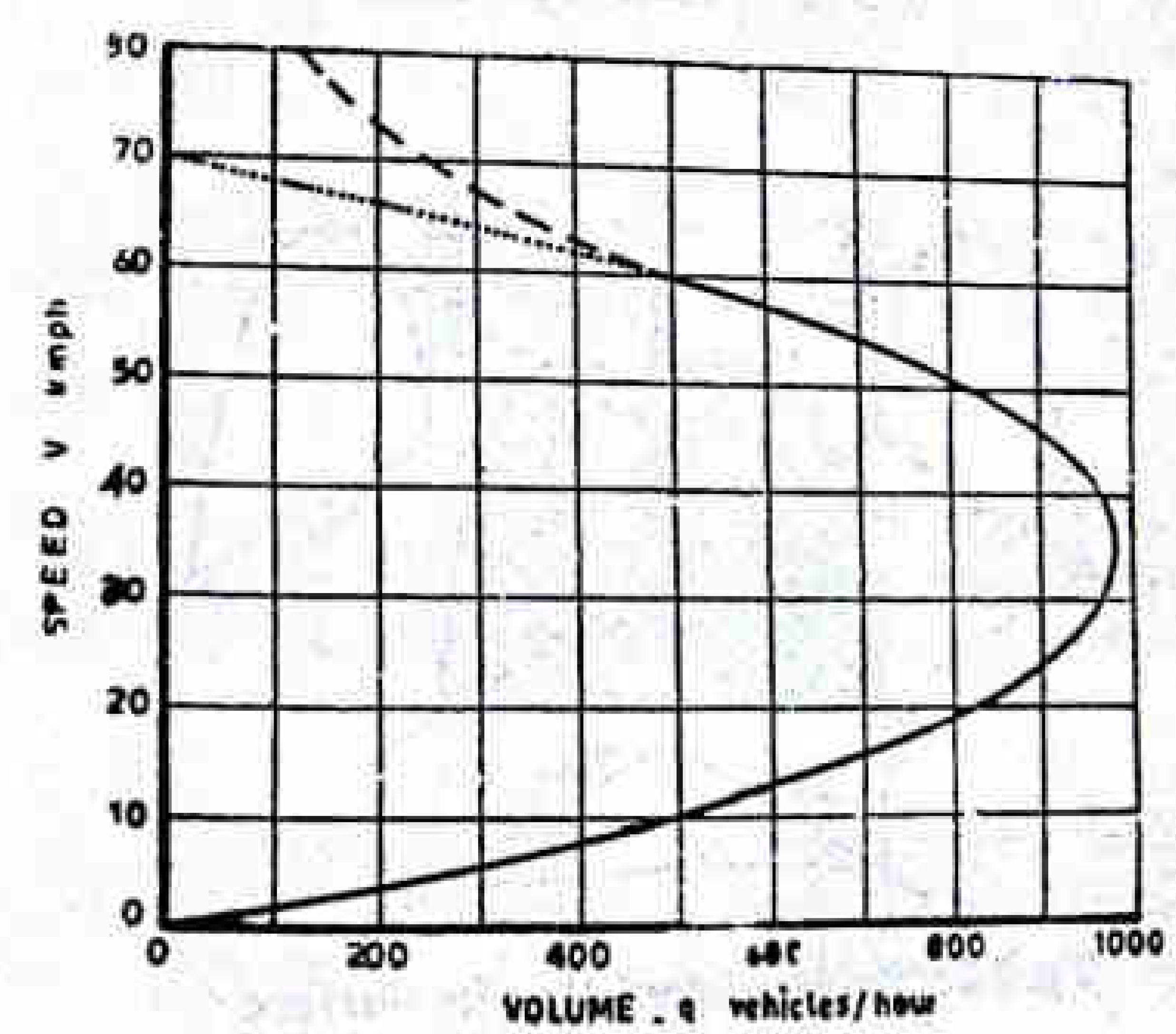


Fig. 5.18 Speed and Volume

The above flow relationships have been established for homogeneous traffic flow. In the case of mixed traffic flow with heterogeneous traffic, these relationships are likely to be quite complex.

Capacity flow

The maximum speed value in Fig. 5.17 and 5.18 is called free means speed V_{sf} and the maximum density at zero speed is called jam density K_j. The maximum flow q_{max} or the capacity flow q_c (see Fig. 5.11) occurs when the speed is $\frac{V_{sf}}{2}$ and density is $\frac{K_j}{2}$ and therefore from Eq. 5.21.

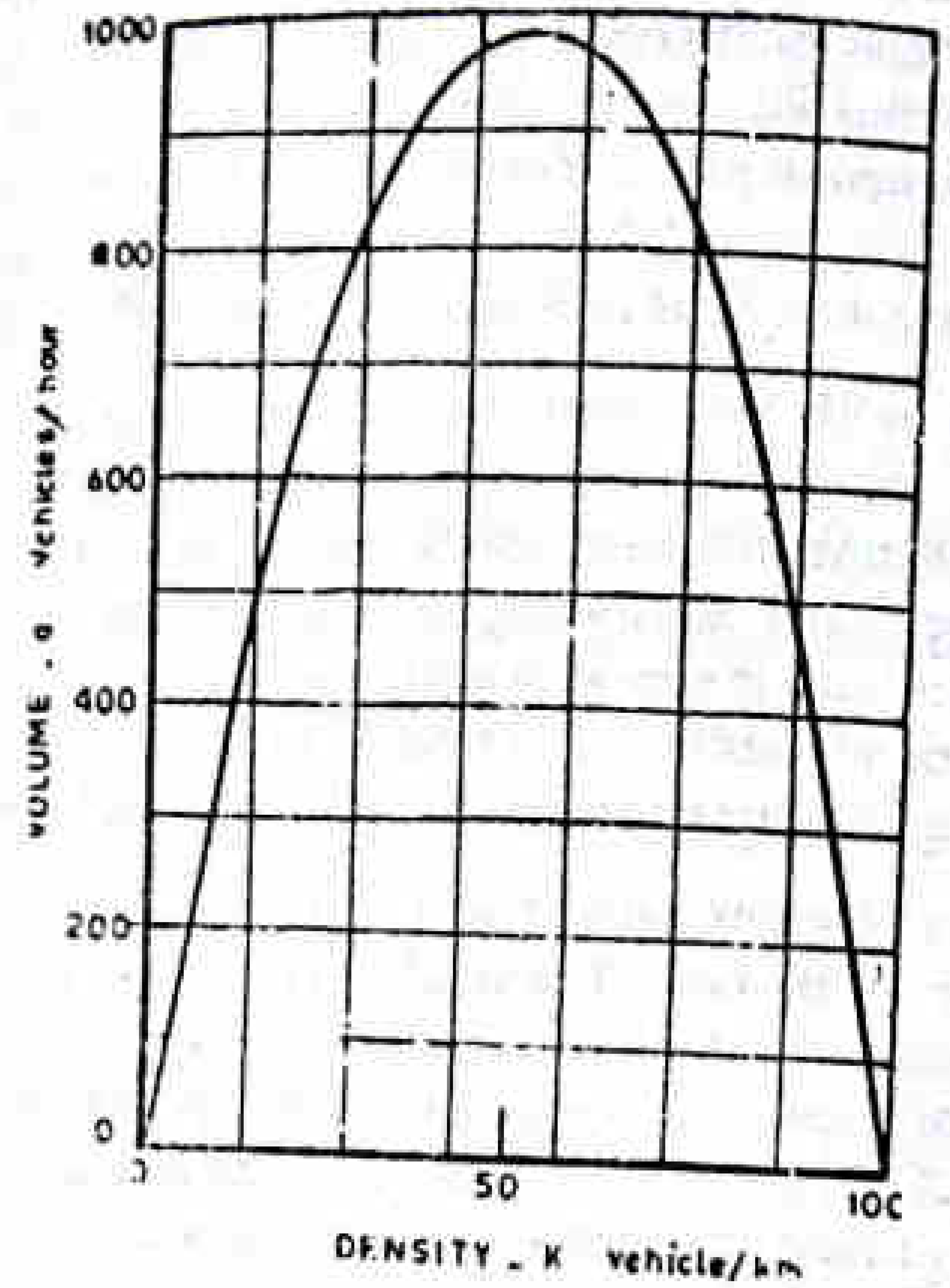


Fig. 5.19 Volume and Density

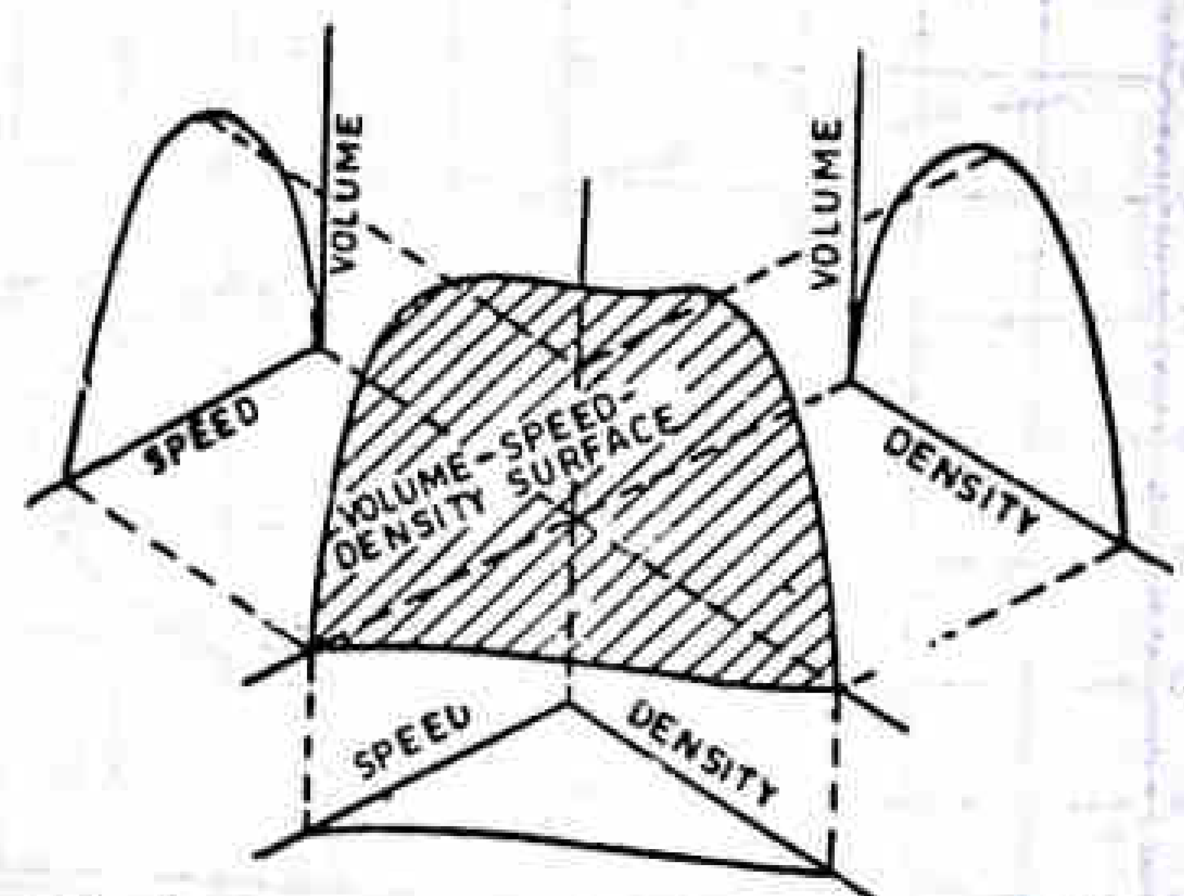


Fig. 5.20 Volume-Speed-Density Surface

$$q_{max} = \frac{V_{sf} \cdot K_j}{4} \quad (5.23)$$

Example 5.12

The free mean speed on a roadway is found to be 80 kmph. Under stopped condition the average spacing between vehicles is 6.9 m. Determine the capacity flow.

Solution

Free mean speed V_{si} = 80 kmph

Jam density $K_j = \frac{1000}{6.9} = 145$ vehicles/km (per lane)

Maximum flow $q_{max} = \frac{80 \times 145}{4} = 2900$ vehicles/hour (per lane)

5.2.5 Future Traffic

The existing traffic operation may be measured by traffic studies as described in Art. 5.2.3. It is necessary to know the future traffic demand in order to design highway facilities adequately. There are different methods for projecting future traffic. These may be broadly classified as : (i) mechanical and (ii) analytical methods.

In the mechanical methods, the past trends of traffic growth are simply projected forward, making use of a percentage increase in various categories of traffic in a region. In some countries it has been estimated that the traffic is doubled in a period of 10 years resulting in the annual growth rate of 7 to 8 percent. In mechanical methods any of the principles such as a correlation index, ratio, analogy composite trend or a growth formula may be used. Correlation index may include gross productivity, national income or fuel consumption. A straight line, geometric progression, compound interest or general growth rate may be considered in the growth formulae.

The analytical methods consider the short term as well as long term policies in estimating the further travel requirements of urban area. It develops analytical expressions to predict travel requirements by relating the social and economic status of the area with the demand for transport. The first comprehensive traffic and transportation plan based on such concept has been prepared for Bombay City to solve urban transport problems, and subsequently studies have been extended to other metropolitan cities of Calcutta, Delhi, Bangalore and Madras.

5.3 TRAFFIC OPERATIONS

In order to have safe traffic operations on roads, it is essential to impose adequate traffic regulations and traffic control devices. It is necessary to impress on the public that these regulations and controls are imposed on the public interest to ensure safety in general.

5.3.1 Traffic Regulations

The traffic regulations should cover all aspects of control of vehicles, driver and all other road users. The regulations should be rational. The following are some of the regulations that are enforced from the point of view of safe traffic operations.

Traffic regulations and laws give legal coverage for strict enforcement. The traffic laws implemented by legislative laws are obligatory on all road users. The laws should however be uniform and clear. Traffic regulations and laws cover the following four phases.

(i) *Driver Controls* : These include driving licenses for light and heavy motor vehicles, driver tests and minimum requirements, financial responsibility and civil liability.

(ii) *Vehicle Controls* : The various regulations and controls on vehicles are vehicle registration, requirements of vehicles, equipment and accessories, maximum dimensions and weight and fitness and inspection of vehicles.

(iii) *Flow Regulations* : Regulations of traffic flow have been laid down such as directions, turning and overtaking, etc. In addition control of vehicle operation in traffic stream are made using appropriate regulatory signs like one-way, speed limit prohibitory signs, pedestrian controls, etc.

(iv) *General Controls* : Some other general regulations and provisions are made to report accidents and recording and disposing traffic violation cases.

The *Motor Vehicle Act of 1939* and the several ordinances appending the Act have covered various traffic regulatory measures in India. The various items covered are issue of driving license, registration of vehicles, transfer of ownerships, distinction between private and public vehicles, transport authorities and inter-state commission, limits of speed, weight, parking and halting places, insurance fees, signs, signals and general provisions for punishment of violations and offences.

One-way streets

In congested streets one of the methods to reduce accidents and to ensure smooth flow of traffic is by regulating traffic along *one-way* streets. The traffic is allowed to move only in one specified direction. Such regulations are possible only when there is a net work of roads connecting two bigger roads so that additional distance to be traversed by some vehicles through these one-way streets is not excessive.

The main advantages of one-way streets may be greater capacity, increased average speed, improved pedestrian movement, and reduction in accident. The various types of conflicts at an intersection are

Crossing Conflicts

Merging Conflicts

Diverging Conflicts

On a right angled road intersection with two-way traffic the total number of conflict points are 24. This consists of 16 crossing conflicts which are the major conflict points. The merging and diverging conflicts are considered as minor conflicts, numbering four each in this case, as shown in Fig. 5.21. If one of the roads is declared as one-way, the conflict points decreases to a total of 11, consisting of seven crossing conflicts and four merging conflicts as shown in Fig. 5.22. When both roads are declared as one-way, there are only four crossing conflicts and two merging conflicts, totaling six as shown in Fig. 5.23.

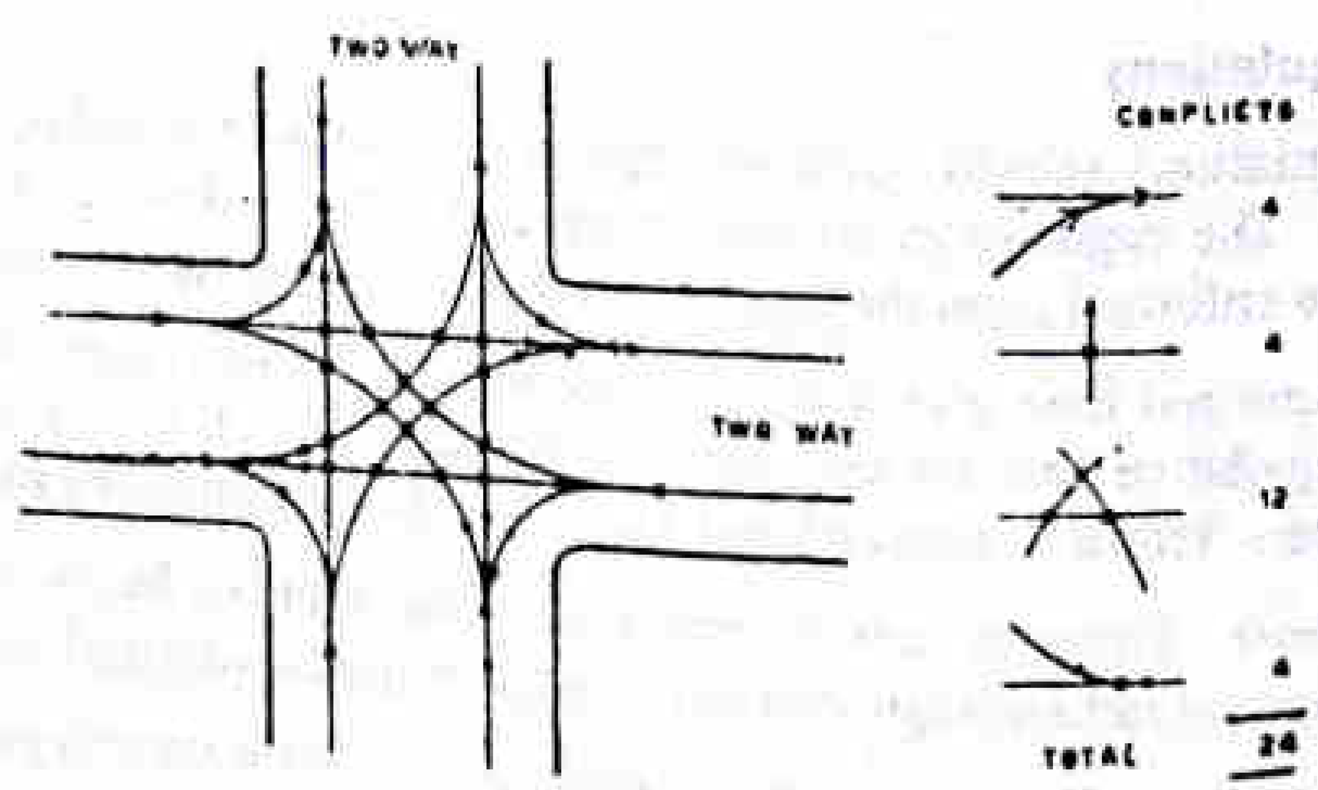


Fig. 5.21 Conflicts with Two-way traffic

Thus the chief advantage of one-way street enforcement is reduction of number of potential points of vehicle conflicts at the uncontrolled intersection as explained above. The potential conflicts and two-way operation and varying number of lanes are given in the following table :

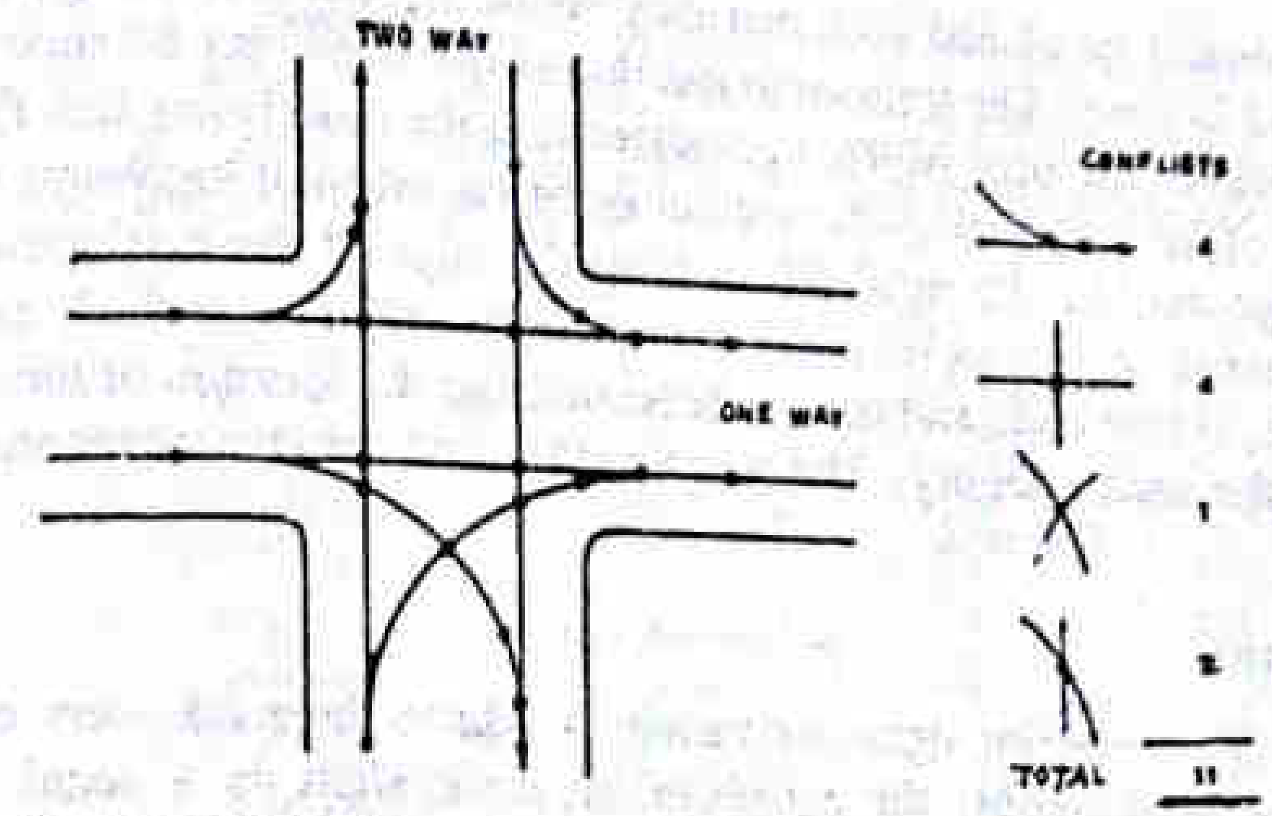


Fig. 5.22 Conflicts with One-way Regulation on One Road

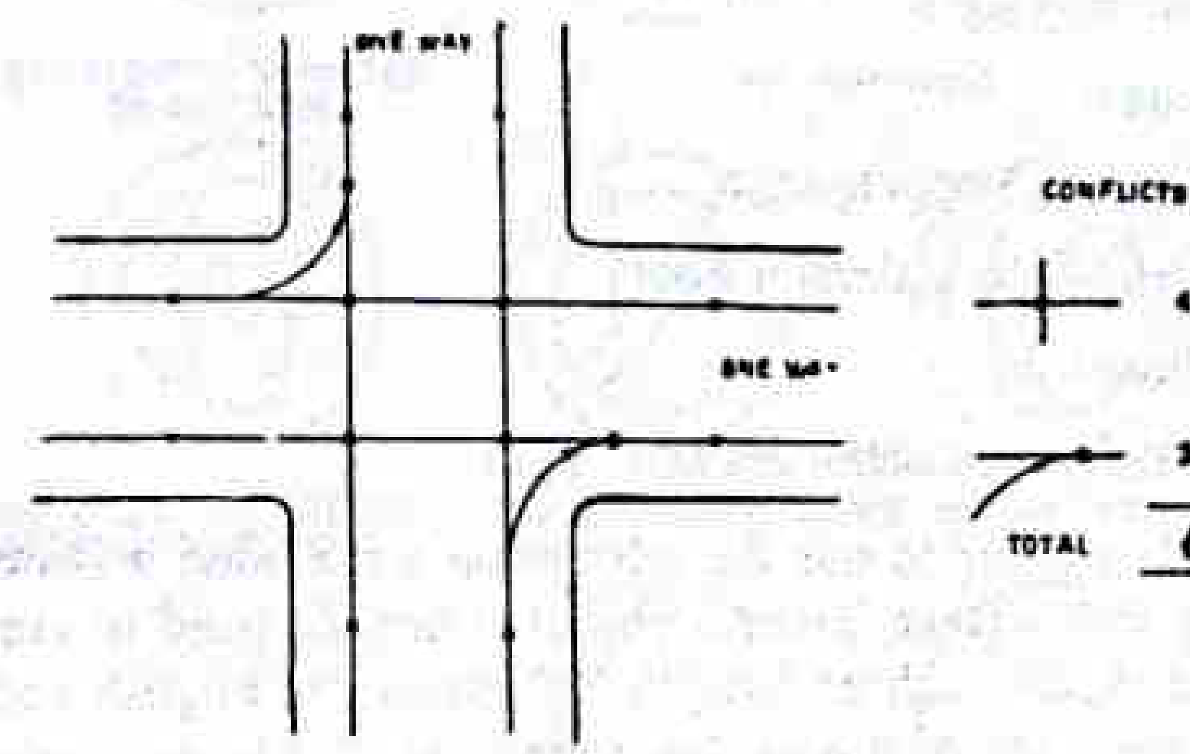


Fig. 5.23 Conflicts with One-way Regulation on Both Roads

Number of lanes*		Number of potential conflicts		
Road A	Road B	Both roads Two-way	A - One-way B - Two-way	Both-roads One-way
2	2	24	11	6
2	3	24	11	8
2	4	32	17	10
3	3	24	13	11
4	4	44	25	18

*3 – lane road when operated as two-way street has one-lane for each direction

5.3.2 Traffic Control Devices

The various aids and devices used to control, regulate and guide traffic may be called traffic control devices. The general requirements of traffic control devices are : attention, meaning, time for response and respect of road users. The most common among these are (a) Signs (b) Signals (c) Markings and (d) Islands. In addition, road lights are useful in guiding traffic during night.

Traffic signs

The traffic signs should be backed by law in order to make them useful and effective. Traffic signs have been divided into three categories according to Indian Motor Vehicles Act. These are (i) Regulatory signs (ii) Warning signs and (iii) Informatory signs.

The signs should be placed such that they could be seen and recognized by the road users easily and in time. The transverse location of the signs may be such that in the case of roads with kerbs, the edge of the sign adjacent to the road is not less than 0.6 m away from the edge of the kerb; on roads without kerbs (as on rural highways with shoulders) the nearest edge may be 2.0 m to 3.0 m from the edge of the carriageway. The signs should be mounted on sign posts painted alternately with 25 cm black and white bands. The size, shape, colour code and the symbols used and the location of the signs should be as specified under each category. The reverse side of all the sign plates should be painted gray.

Regulatory signs

Regulatory or mandatory signs are meant to inform the road users of certain laws, regulations and prohibitions; the violation of these signs is a legal offence. The regulatory signs are classified under the following sub-heads :

(i) Stop and Give-way signs

(ii) Prohibitory signs

(iii) No Parking and No Stopping signs

(iv) Speed Limit and Vehicle Control signs

(v) Restriction Ends sign

(vi) Compulsory Direction Control and other signs

The stop sign is intended to stop the vehicles on a roadway; it is octagonal in shape and red in colour with a white border. This sign may be used in combination with a rectangular definition plate with the word 'STOP' written in English and other languages as necessary. The give way sign is used to control the vehicles on a road so as to assign right of way to traffic on other roadways. This sign is triangular in shape with the apex downwards and white in colour with a red border; this sign may also be used in combination with a definition plate. These signs are shown in Fig. 5.24.

Prohibitory signs are meant to prohibit certain traffic movements, use of horns or entry of certain vehicle class. These signs are circular in shape and white in colour with a red border. The common prohibitory signs are, Straight Prohibited, No Entry, One-way, Vehicles Prohibited in Both Directions, All Motor Vehicles Prohibited, Truck Prohibited, Bullock Cart and Hand Cart Prohibited, Bullock Cart Prohibited, Tonga Prohibited, Hand Cart Prohibited, Cycle Prohibited, Pedestrian prohibited, Right/Left Turn Prohibited, U-Turn Prohibited, Overtaking Prohibited and Horn Prohibited.

No parking sign is meant to prohibit parking of vehicles at that place, the definition plate may indicate the parking restriction with respect to days, distance, etc. The No Parking sign is circular in shape with a blue back ground, a red border and an oblique red bar at an angle of 45 degrees. No Stopping/Standing sign is meant to prohibit stopping of vehicles at that place; the scope of the prohibition may be indicated on a definition plate. The No Stopping/Standing sign is circular in shape with blue back ground, red border and two oblique red bars at 45 degrees and right angle to each other. The sketches of the Prohibitory Signs, No Parking and No Stopping signs are shown in Fig. 5.24.

Speed Limit signs are meant to restrict the speed of all or certain classes of vehicles on a particular stretch of a road. These signs are circular in shape and have white back ground, red border and black numerals indicating the speed limit. The Vehicle Control

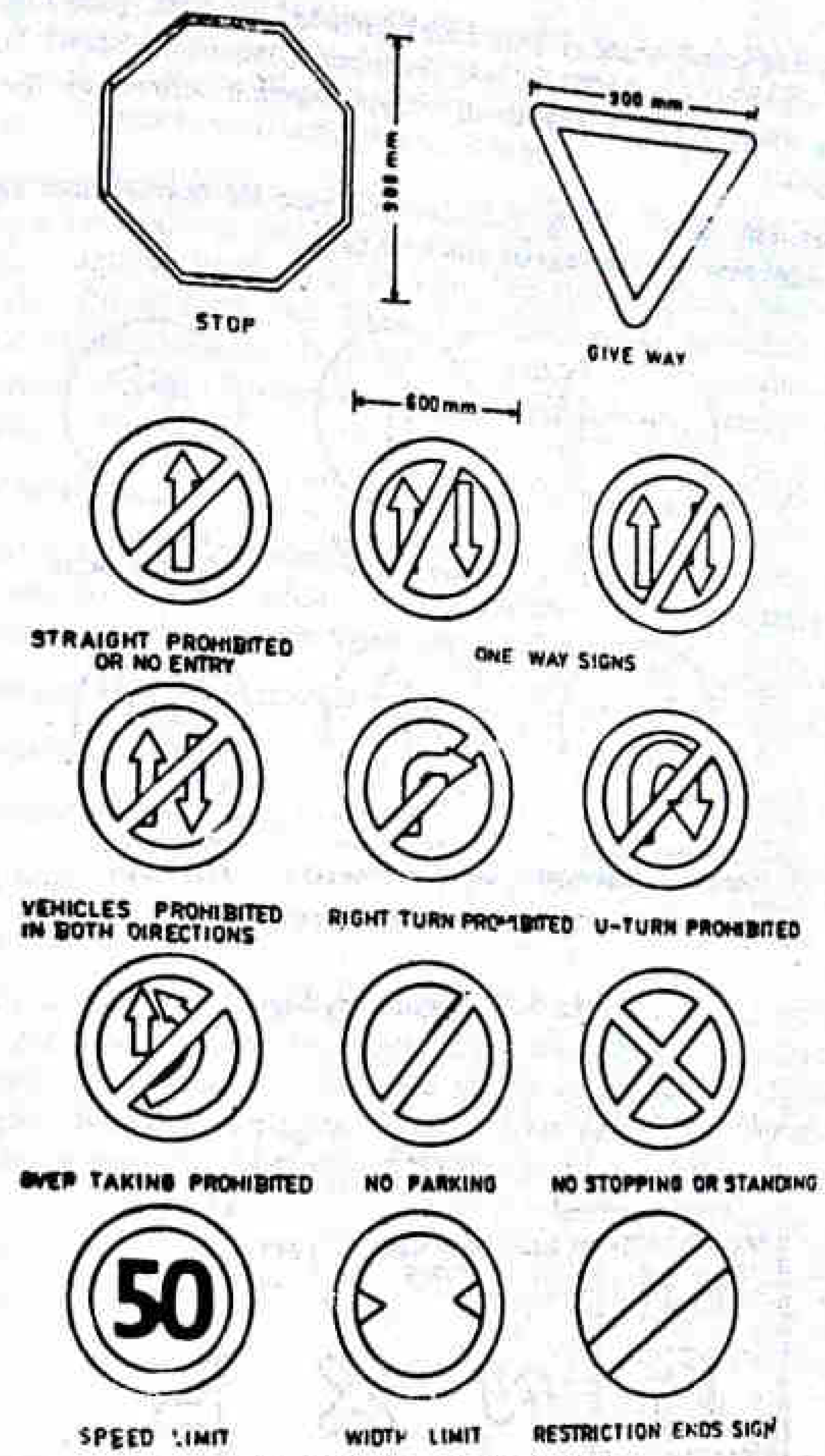


Fig. 5.24 Regulatory Signs

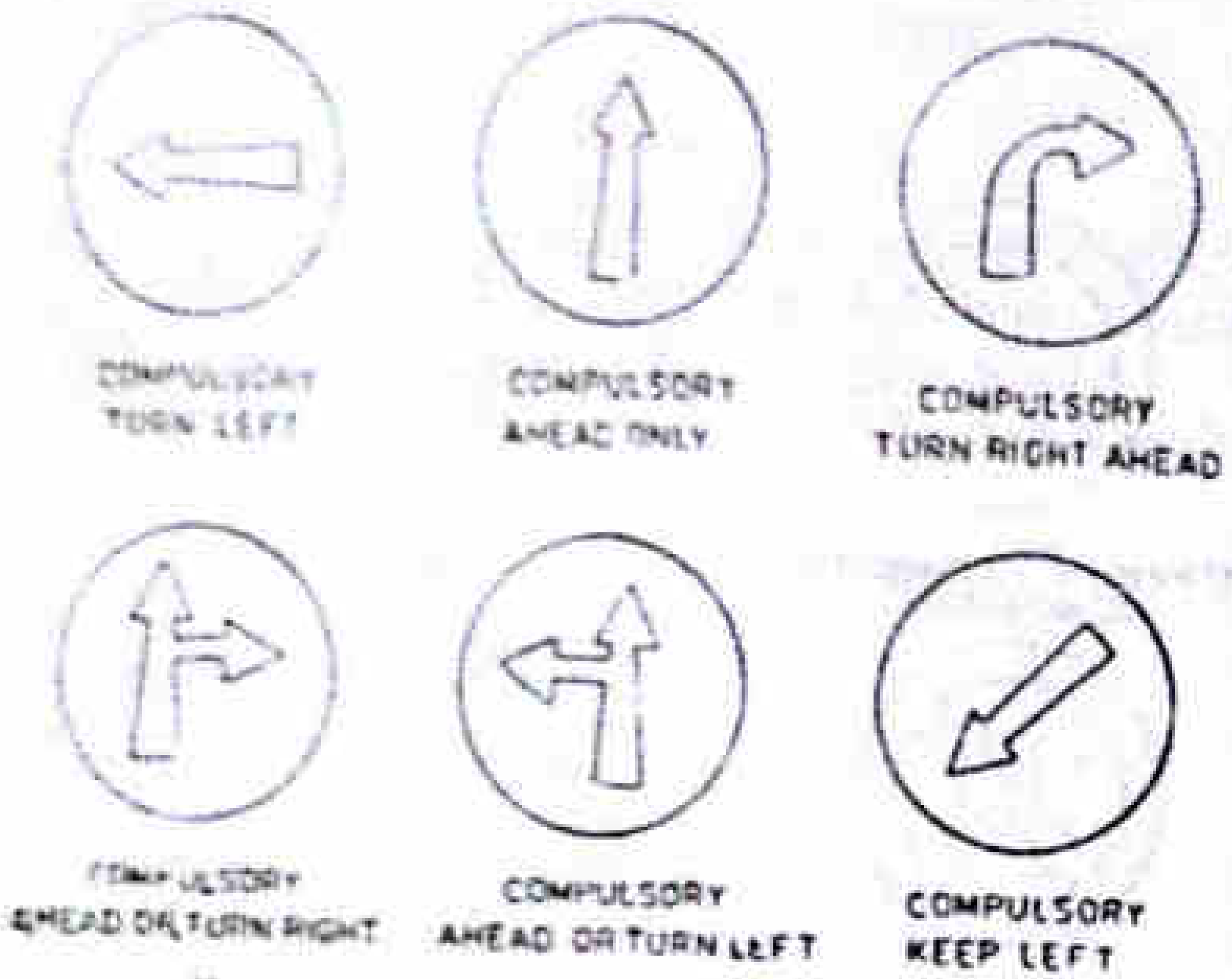
Sings are also similar to Speed Limit signs with black symbols instead of the numerals. The common controls are Width Limit, Height Limit, Length Limit, Load Limit and Axle Load Limit. The definition plate may be used in combination to give more details, symbolically or by words.

Restriction Ends sign indicates the point at which all prohibitions notified by prohibitory signs for moving vehicles case to apply. These signs are also circular with a white back ground and a broad diagonal black band at 45 degrees.

Compulsory Direction Control signs indicate by arrows, the appropriate directions in which the vehicles are obliged to proceed, or the only directions in which they are permitted to proceed. These signs are circular in shape with a blue back ground and white direction arrows. Some of the Compulsory Direction Controls are Compulsory

Turn Left, Ahead Only, Ahead or Turn Left Right and keep Left. (See Fig. 5.25). Other Compulsory signs are Compulsory Cycle Track and Compulsory Sound Horn. These are indicated by white symbols instead of White direction arrows of the Compulsory Direction Signs.

The dimensions shown in Fig. 5.24 and 5.25 are for normal size signs; however smaller size signs may be permitted on minor roads.



(Compulsory Direction Control Signs)

Fig. 5.25 Regulatory Signs

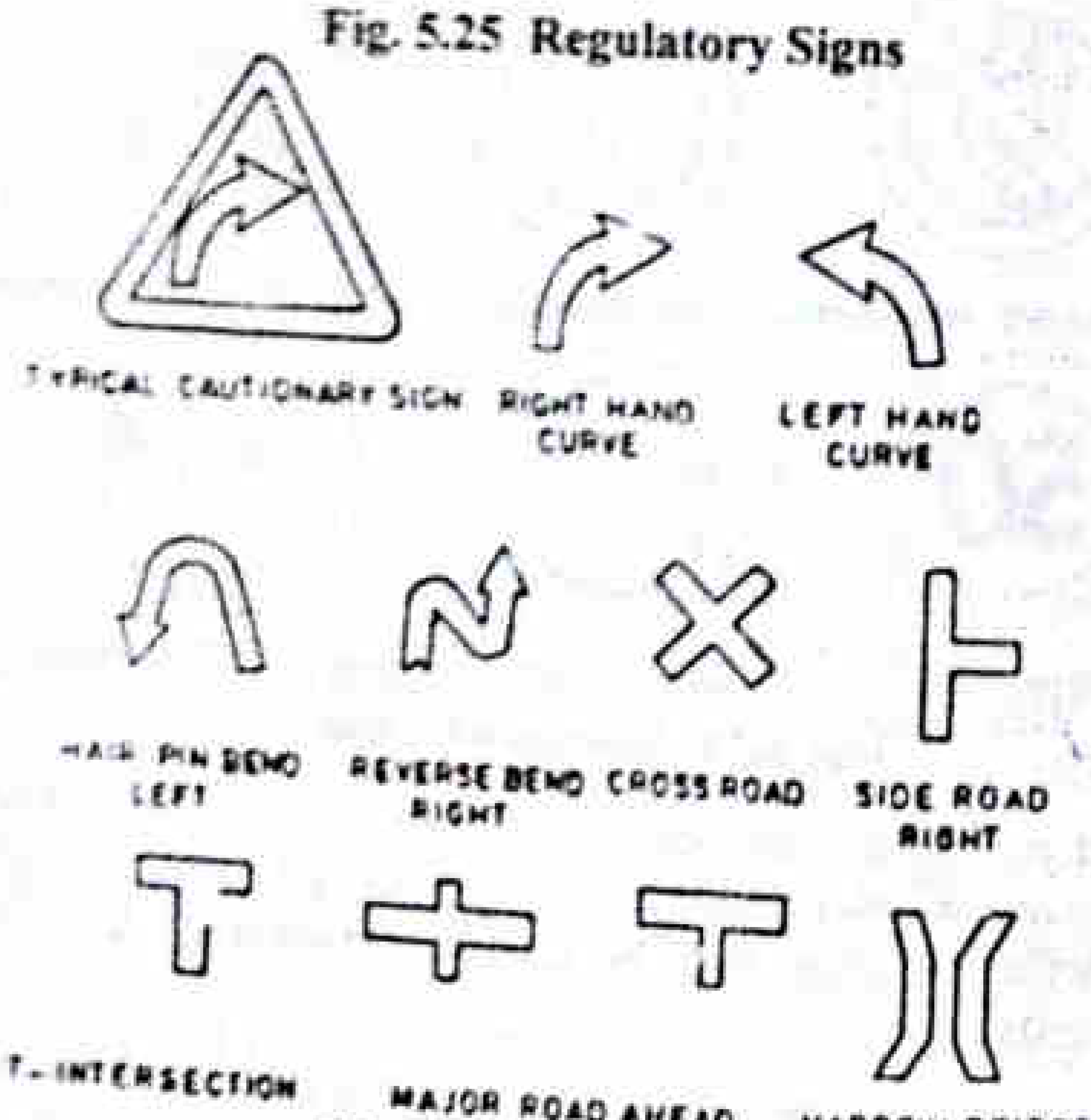


Fig. 5.26 Warning Signs

Warning Signs

Warning or Cautionary signs are used to warn the road users of certain hazardous conditions that exist on or adjacent to the roadway. The Warning signs are in the shape of equilateral triangle with its apex pointing upwards. They have a white back ground, red border and black symbols. The warning signs are to be located in advance of the hazard warned against. [illegible] respectively on National/State Highways, Major District Roads, Other District Roads and Village Roads, on urban roads this distance is 50 metre.

The commonly used Warning signs are, Right Hand/Left Hand Curve, [illegible] Pin Bend, Right/Left Reverse Bend, Steep Ascent/Descent, Narrow Bridge/Road, [illegible] Gap in Median, Slippery Road, Cycle Crossing, Pedestrian Crossing, School Zone, Men at Work, Ferry, Cross Road, Side Road, T-Intersection, Y-Intersection, Major Road Ahead, Round About, Dangerous Dip, Hump or Rough Road, Barrier Ahead, Unguarded Railway Crossing, Guarded Railway Crossing and Falling Rock. Some of these Warning Signs are shown in Fig.5.25.

Informatory sings

These signs are used to guide the road users along routes, inform them of destination and distance and provide with information to make travel easier, safe and pleasant. The information signs are grouped under the following sub-heads :

(i) Direction and Place Identification signs

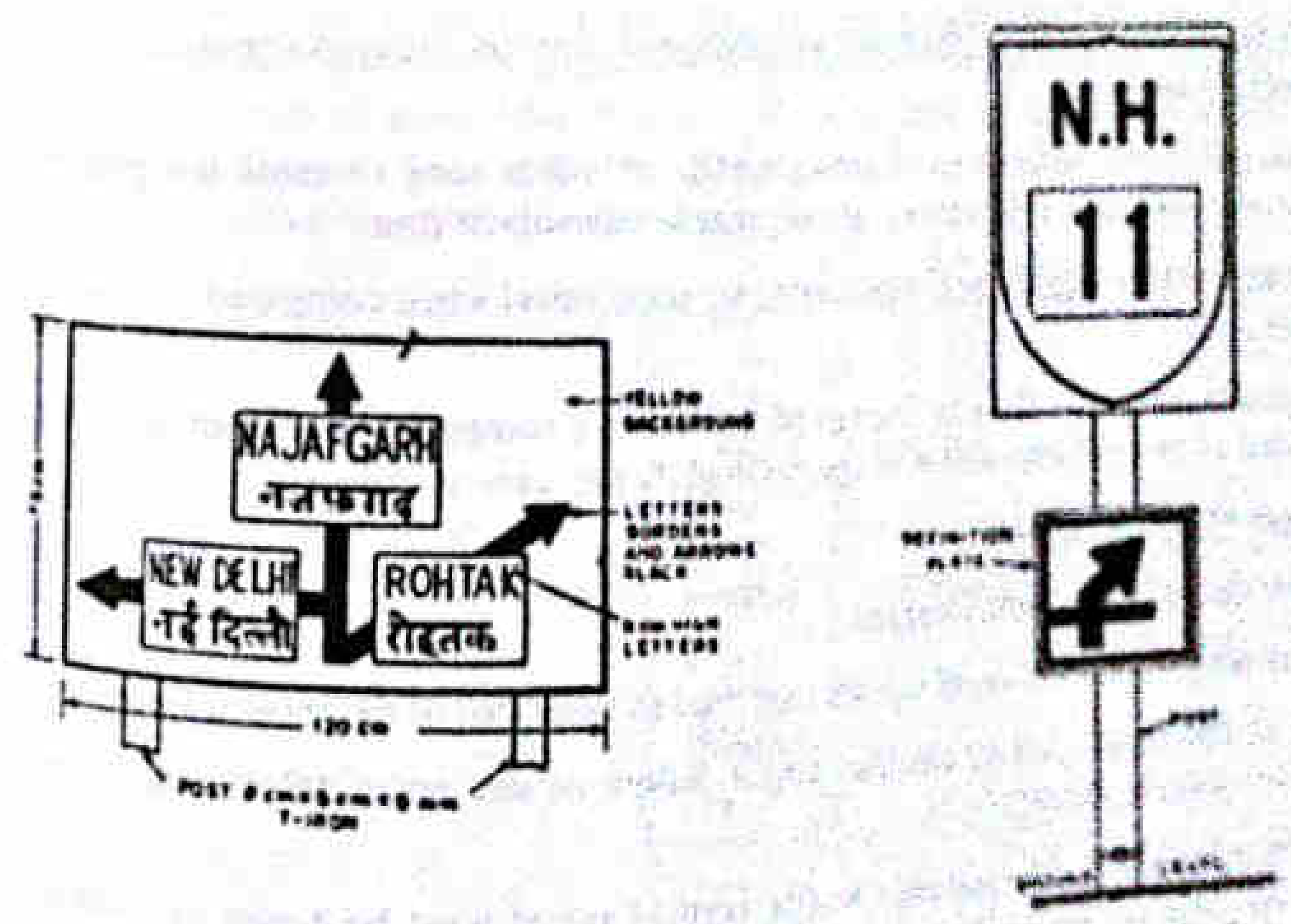
(ii) Facility Information signs

(iii) Other Useful Information signs

(iv) Parking sings

(v) Flood Gauge

The Direction and Place Identification signs are rectangular with white back ground, black border and black arrows and letters. The inscriptions should be in English and other languages as necessary. The signs of this group include Destination signs, Direction signs, Re-assurance signs, Route Marker and Place Identification signs. Figure 5.27 shows some of the Informatory signs.



(a) Road Junction Approach

(b) Route Marker Sign

Fig. 5.27 Informatory Signs

The Facility Information signs are rectangular with blue back ground and white/black letters/symbols. Some of these signs indicate Public Telephone, Petrol Pump, Hospital, First Aid Post, Eating Place and Resting Place. Other useful information signs include No Through Road, No Through Side Road, etc. Parking signs are set up parallel to the road using square sign board with blue black ground and white coloured letter 'P'. Additional definition plate may be used to indicate category of vehicle for which parking space is reserved, direction of parking space etc.

Flood Gauge sign should be installed at all cause ways and submersible bridges or culverts to indicate to the road users the height of the flood above road level.

Traffic signals

At intersection where there are a large number of crossing and right-turn traffic, there is possibility of several accidents as there cannot be orderly movements. The earlier practice has been to control the traffic by means of traffic police by showing stop signs alternately at the cross roads so that one of the traffic streams may be allowed to move while the cross traffic is stopped. Thus the crossing streams of traffic flow are separated by time, seggregation. Traffic signals are control devices which could alternately direct the traffic to stop and proceed at intersections using red and green traffic light signals automatically. The main requirements of traffic signal are to draw attention, provide meaning and time to respond and to have minimum waste of time.

Advantages of traffic signals

Properly designed traffic signals have the following uses :

(i) They provide orderly movement of traffic and increase the traffic handling capacity of most of the intersections at grade.

(ii) They reduce certain types of accidents, notably the right angled collisions.

(iii) Pedestrians can cross the roads safely at the signalised intersection.

(iv) The signals allow crossing of the heavy traffic flow with safety.

(v) When the signal system is properly co-ordinated, there is a reasonable speed along the major road traffic.

(vi) Signals provide a chance to crossing traffic of minor road to cross the path of continuous flow of traffic stream at reasonable intervals of time.

(vii) Automatic traffic signal may workout to be economical when compared to manual control.

(viii) The quality of traffic flow is improved by forming compact platoons of vehicles, provided all the vehicles move at approximately the same speed.

Disadvantages of traffic signals

(i) The rear-end collisions may increase.

(ii) Improper design and location of signals may lead to violations of the control system.

(iii) Failure of the signal due to electric power failure or any other defect may cause confusion to the road users.

The decision to install an automatic traffic control signal must be based on careful analysis of the existing traffic data and on sound engineering judgment. The major emphasis in the criteria for signal control is the volume of traffic entering the intersection and its crossing movements.

The various terms used in traffic signals are briefly explained here. The period of time required for one complete sequence of signal indications is called cycle. A part of the signal cycle allocated to a traffic movement or a combination of traffic movement is called *phase*. Any of the division of the signal cycle during which signal indications do not change is called the *interval*. The engineer has to design the signal with the sequence and duration of individual phases to serve all approaching traffic at a desired *level of service*. The level of service is measured by the vehicle delay, the *queue length* or the number of vehicle backed up and the probability of a vehicle entering the intersection during the first green phase after arrival.

The capacity of a signalised intersection depends on physical factors of the roads such as roadway width, number of lanes geometric design of intersection and also the green and red phases of the traffic signal. In addition, the capacity is affected by operational and control factors such as number of turning movement, number and size of commercial vehicles, pedestrian traffic, peak hour demands, parking regulations, turn control, traffic signal characteristics and abutting land use.

Type of traffic signals

The signals are classified into the following types :

(i) Traffic control signals

(a) Fixed-time signal

(b) Manually operated signal

(c) Traffic actuated (automatic) signal

(ii) Pedestrian signal

(iii) Special traffic signal

The traffic control signals have three coloured light glows facing each direction of traffic flow. The red light is meant for *Stop*, the green light indicates *Go* and the amber or yellow light allows the *clearance time* for the vehicles which enter the intersection area by the end of green time, to clear off. A typical signal head is shown in Fig. 5.28. Additional signals showing green lights for separate movements of turning traffic may also be provided where necessary.

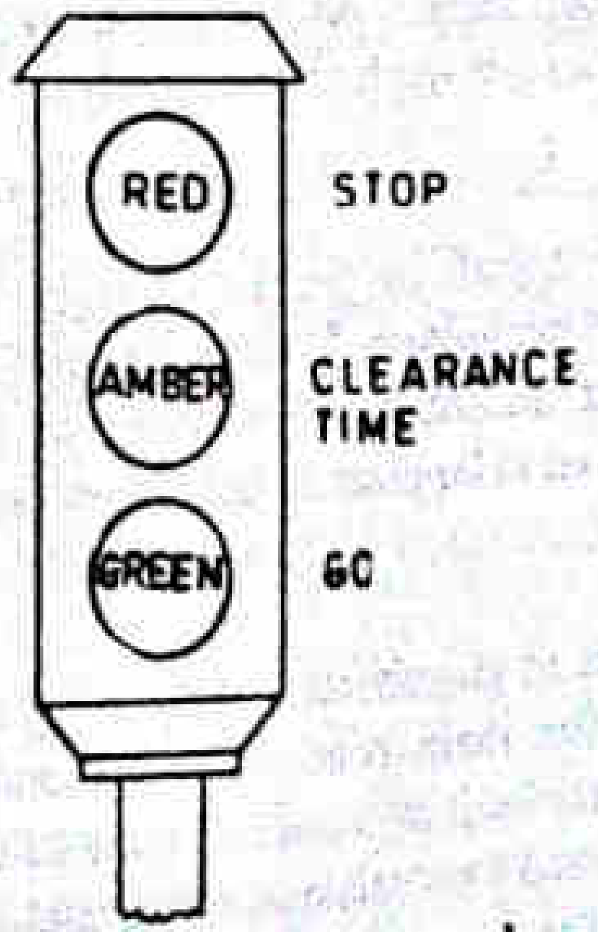


Fig. 5.28 Traffic Signal

Fixed-time signal or pre-timed signals are set to repeat regularly a cycle of red, amber and green lights. The timing of each phase of the cycle is predetermined based on the traffic studies and they are the simplest type of automatic traffic signals which are electrically operated. The main draw back of the signal is that some times the traffic flow on one road may be almost nil and traffic on the cross road may be quite heavy. Yet as the signal operates with fixed timings, the traffic in the heavy stream will have to stop at red phase.

Traffic actuated signals are those in which the timings of the phase and cycle are changed according to traffic demand. In semi-actuated traffic signals the normal green phase of an approach may be extended upto a certain period of time for allowing a few more vehicles approaching closely, to clear off the intersection with the help of detectors installed at the approaches. In fully actuated traffic signals the detectors and a computer assigns the right of way for various traffic movements on the basis of demand and pre-determined programming. But these are very costly to be installed at all intersections.

In some cities in India the traffic police are assigned the duty to watch the traffic demand from suitable observation point during the peak hours on various approaches and to vary the timings of the phases and cycle according to thé actual traffic demand.

When there are series of signals on a city road at each intersection with cross road, the signal system may be operated with only one controller. But it is desirable that a vehicle moving along a main road at normal speed should not have to a stop at every signalised intersection till getting the "go" signal. Hence there should be proper *Co-ordination* of the signal system to provide a *through band.*

Type of traffic signal system

There are four general types of co-ordination of signals for road network, as listed below :

- Simultaneous system
- Alternate system
- Simple progressive system, and
- Flexible progressive system

Simultaneous System

In this system all the signals along a given road always show the same indication (green, red etc.) at the same time. As the division of cycle is also the same at all intersections, this system does not work satisfactorily.

Alternate System

In this system, alternate signals or groups of signals show opposite indications in a route at the same time. This system is also operated by a single controller, but by reversing the red and green indicator connections at successive signal systems. This system generally is considered to be more satisfactory than the simultaneous system.

Simple Progressive System

A time schedule is made to permit, as nearly as possible, a continuous operation of groups of vehicles along the main road at a reasonable speed. The signal phases controlling "Go" indications along this road is scheduled to work at the predetermined time schedule. The phases and intervals at each signal installation may be different; but each signal unit works as fixed time signal, with equal signal cycle length.

Flexible Progressive System

In this system it is possible to automatically vary the length of cycle, cycle division and the time schedule at each signalized intersection with the help of a computer. This is the most efficient system of all the four types described above.

Pedestrian signal are meant to give the right of way to pedestrians to cross a road during the 'walk period' when the vehicular traffic shall be stopped by red or stop signal on the traffic signals of the road.

Flashing beacons are meant to warn the traffic. At flashing red signals, the drivers of vehicles shall stop before entering the nearest cross walk at an intersection or at a stop line, when marked Flashing yellow signals are caution signals meant to signify that drivers may proceed with caution.

Warrants for traffic control signal installation

Traffic control signals should not be installed unless one or more of the following signal warrants are met. The necessary data should be collected by means of traffic-engineering studies :

(i) Minimum vehicular volume warrant specifies that the average traffic volume for eight hours on both approaches should be atleast 650 motor vehicles per hour on major streets with single lane and 800 vehicles on the streets with two or more lanes. Further the number of motor vehicles approaching the intersection on minor street (on one direction only) is atleast 200 vehicles per hour on single lane street and 250 vehicles per hour when there are two or more lanes. However when the average approach speed or the 85th percentile speed on major street exceeds 60 kmph or when the intersection lies within built-up area, the vehicular volume warrant may be decreased to 70 percent of the above requirement.

(ii) Interruption of continuous traffic flow on the major street with 1000 to 1200 vehicles per hour that there is undue delay or hazard to traffic on minor road with a traffic of 100 to 150 vehicles per hour in one direction only during any eight hours of an average day.

(iii) Minimum pedestrian volume warrant of 150 or more pedestrians per hour cross a major street with over 600 vehicles per hour on both approaches, (1000 vehicles per hour in the case of main street with raised median). However when the average approach speed or the 85th percentile speed exceed 60 kmph, 70 percent of the above requirements may be adopted.

(iv) Accident experience warrant showing that other measures have failed to decrease the accident frequency or when five or more accidents (involving over Rs. 2000 due to injury and damage) have occurred within 12 months period. However signal installation should not seriously disrupt the traffic flow.

(v) Combination of warrants when no single warrant is satisfied but indicating two or more warrants of (i), (ii) or (iii) above are satisfied to the extent of 80 percent or more of the stated volume.

Design of isolated fixed time signal

In the design of a signalised intersection, the objective should be to provide sufficient capacity for the volume of traffic approaching the intersection. The design should aim at minimising total delay, building short queues, and providing a high probability of passing through the intersection on the first given period for most users. Signal timing should be in accordance with traffic flow on intersection. The cycle lengths are normally 40 to 60 seconds for two phase signal. Longer cycle lengths are in use for complex traffic flow and for more than two phases.

General Principles of Signal Design

(i) Stop time or red phase R1 of a signal is the sum of go and clearance intervals or green and amber phases for the cross flow i.e., $G_2 + A_2$ at a two phase signal. During this interval, the pedestrian crossing time may also be incorporated for the road, if turning movements are not permitted.

(ii) Towards the end of red phase, there may be a short duration when the amber lights are put-on along with red light signal in order to indicate 'get set' to go. This phase is the last part of red phase itself and may be called 'red-amber' or 'initial amber'. The vehicles are not supposed to cross the stop line during the red amber period.

(iii) Clearance time or clearance amber phase is provided just after the green phase before the red phase, to fulfil two requirements :

(a) Stopping time for approaching vehicle to stop at stopline after the signal changes from green to amber and not to cross the line by the time the signal changes to red phase.

(b) Clearance time for the vehicle which is approaching the stop line at legal speed while the signal changes from green to amber, allowing sufficient time for the vehicle to cross the intersection area as it may not be possible for the vehicle to stop before the stop line at that stage. Usually 2.0 to 4.0 seconds would be suitable for the amber phase.

(iv) Go time or green time is decided based on the approach volume during peak hour and to enable the queued vehicles to clear off in most of the cycles.

Two approximately design procedures (viz., trial cycle method and approximate method) and one rational approach (Webster's method) for the design of traffic signal cycles are given below for fixed time traffic signals at cross roads. In addition the signal design method as per the guidelines of the IRC is also given. For the purpose of simplicity, two phase traffic signals with no turning movements are illustrated here. The methods may be suitably extended for multiphase operation.

(1) Trial cycle method

The 15 minute-traffic counts n_1 and n_2 on road 1 and 2 are noted during the design peak hour flow. Some suitable trial cycle C_1 second is assumed and the number of the assumed cycles in the 15 minutes or 15 × 60 seconds period is found to be $(15 \times 60)/C_1$ i.e. $(900/C_1)$. Assuming an average time headway 2.5 seconds, the green periods G_1 and G_2 of roads 1 and 2 are calculated to clear the traffic during the trial cycle.

$$G_1 = \frac{2.5\, n_1 C_1}{900} \text{ and } G_2 = \frac{2.5\, n_2 C_2}{900}$$

The amber periods A_1 and A_2 are either calculated or assumed suitably (3 to 4 seconds) and the length C_1' is calculated, equal to $(G_1 + G_2 + A_1 + A_2)$ seconds. If the calculated cycle length C_1' works out to be approximately equal to the assumed cycle length C_1, the cycle length is accepted as the design cycle. Otherwise the trials are repeated till the trial cycle length works out approximately equal to the calculated value.

Example 5.13

The 15 minute-traffic counts on cross roads 1 and 2 during peak hour are observed as 178 and 142 vehicles per lane respectively approaching the intersection in the direction of heavier traffic flow. If the amber times required are 3 and 2 seconds respectively for two loads based on approach speeds, design the signal timings by trial cycle method. Assume an average time headway of 2.5 seconds during green phase.

Solution

Trial (i)

Assume a trial cycle $C_1 = 50$ secs.

Number of cycles in 15 mins $= \frac{900}{50} = 18$

Green time for road 1, allowing an average time headway of 2.5 secs. per vehicle

$$G_1 = \frac{178 \times 2.5}{18} = 24.7 \text{ secs.}$$

Green time for road 2, $G_2 = \frac{142 \times 2.5}{18} = 19.7$ secs.

Amber times A_1 and A_2 are 3 and 2 secs. (given)

Total cycle length $= 24.7 + 19.7 + 3.0 + 2.0 = 49.4$ secs.

As this is lower than the assumed trial cycle of 50 secs., another lower cycle length may be tried.

Trial (ii)

Assume trial cycle $C_2 = 40$ secs.

Number of cycles in 15 minutes $= \frac{900}{40} = 22.5$

Green time for road 1, $G_1 = \frac{178 \times 2.5}{22.5} = 19.8$ secs.

Green time for road 2, $G_2 = \frac{142 \times 2.5}{22.5} = 15.8$ secs.

Total cycle length $= 19.8 + 15.8 + 3 + 2 = 40.6$ secs.

Trial (iii)

Assume trial cycle $C_3 = 45$ secs.

Number of cycles in 15 min period $= \frac{900}{45} = 20$

Green time for road 1, $G_1 = \frac{178 \times 2.5}{20} = 22.25$ secs.

$$\text{Green time for road 2, } G_2 = \frac{142 \times 2.5}{20} = 17.75 \text{ secs.}$$

$$\text{Total cycle length} = 22.25 + 17.75 + 3 + 2 = 45.0 \text{ secs.}$$

Therefore the trial cycle of 45 secs. may be adopted with the following signal phases:

$G_1 = 22.25$, $G_2 = 17.75$, $A_1 = 3.0$, $A_2 = 2.0$ and Cycle length $= 45.0$ secs.

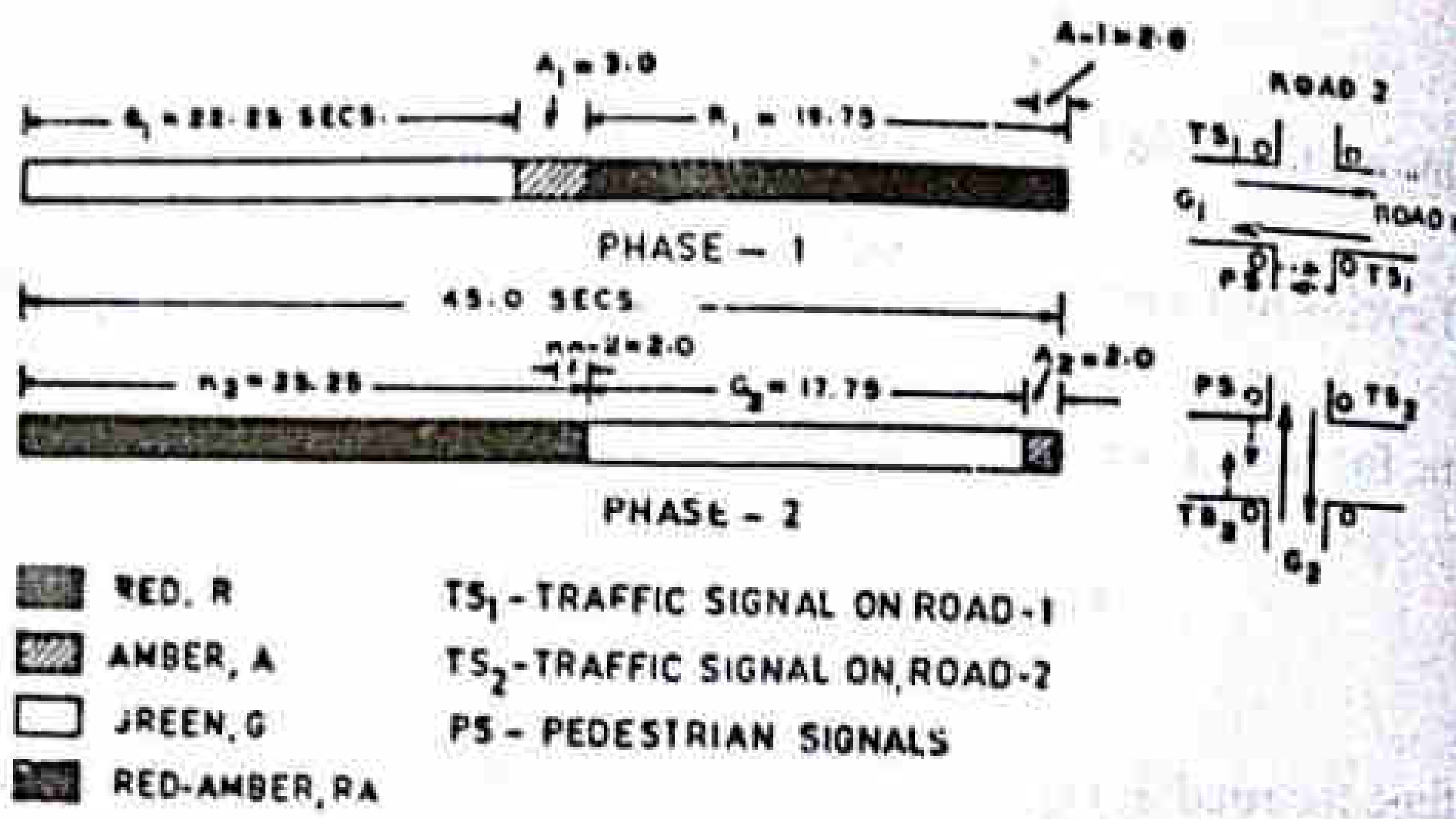


Fig. 5.29 Phase Diagram and Details of Signal Setting (Example 5.13)

(2) Simple design of pedestrian and traffic signals by approximate method

The following design procedure is suggested for the simple design of a two phase signal unit at cross roads, along with pedestrian signals :

(i) Based on approach speeds of the vehicles, the suitable, clearance interval between green and red period i.e., clearance amber periods are selected. The amber periods may be taken as 2, 3 and 4 seconds for low, medium and fast approach speeds.

(ii) Based on pedestrian walking speed of 1.2 m per second, the clearance for pedestrian time is also calculated.

(iii) Minimum red time of traffic signal is taken as pedestrian clearance time for crossing plus initial interval for pedestrians to start crossing. This red time is equal to the minimum green time plus amber time for the cross road.

(iv) The minimum green time is calculated based on pedestrian criterion, equal to pedestrian clearance time for cross road plus an initial interval when pedestrians may start to cross minus amber period. This is equal to red time for cross road minus amber period for the cross road.

(a) with pedestrian signal the initial interval is the WALK period; this should not be less than seven seconds.

(b) where no pedestrian signal is used, a minimum period of five seconds is used as initial interval.

(v) The actual green time needed is then increased based on the ratio of approach volume for the heaviest traffic volume per hour per lane. The cycle length so obtained is adjusted for the next higher 5-second interval. The extra time is then distributed to green timings in proportion to the approaching volumes of traffic.

(vi) The values so obtained are calculated on percentage basis if the controller settings are in per cent of cycle.

(vii) The timings so obtained are installed in the controller and the operations are then observed at the site during peak traffic hours. Corrections or modifications are carried out if needed.

The design of a simple two-phase signal is illustrated by an example below.

Example 5.14

An isolated signal with pedestrians indication is to be installed on a right angled intersection with road A 18 m wide and road B, 12 m wide. The heaviest volume per hour for each lane of road A and road B are 275 and 225, respectively. The approach speeds are 55 and 40 kmph, for A and road B respectively. Design the timings of traffic and pedestrian signals.

Solution

The layout of traffic and pedestrian signals is shown in Fig. 5.30.

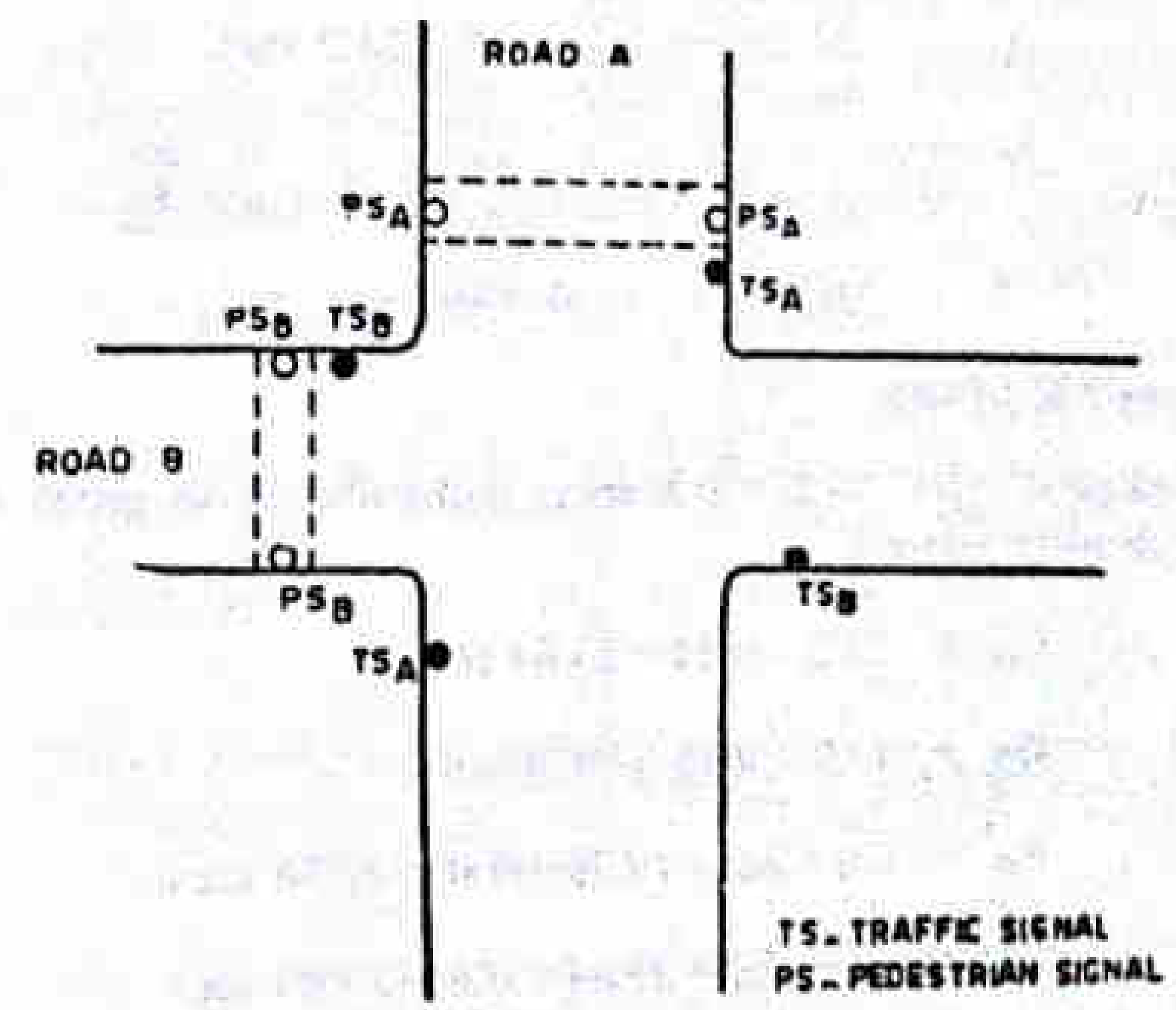


Fig. 5.30 Placement of Signals (Example 5.14)

Design of traffic signals

(i) Based on the approach speed; amber periods :

For road A with 55 kmph, amber period, $A_A = 4$ secs.

For road B with 40 kmph, amber period, $A_B = 3$ secs.

(ii) Based on the pedestrian walking speed of 1.2 m/sec, the pedestrian clearance time is calculated.

$$\text{Road A} = \frac{18}{1.2} = 15 \text{ seconds;}$$

$$\text{Road B} = \frac{12}{1.2} = 10 \text{ seconds}$$

(iii) Adding 7 secs. for initial walk-period, minimum red time for road A is (15 + 7) secs. and that for road B is (10 + 7) secs.

(iv) Minimum green times based on pedestrian criterion :

$$\text{Road B} = 15 + 7 - 3 = 19 \text{ secs.}$$

$$\text{Road A} = 10 + 7 - 4 = 13 \text{ secs.}$$

(v) Based on approach volume, the green time calculated is increased for Road A with higher traffic volume.

Use relation $\frac{G_A}{G_B} = \frac{n_A}{n_B}$

G_A and G_B are green times and n_A and n_B are approach volume per lane

G_B is taken as 19 seconds as in (iv) above.

Green time for Road A,

$$G_A = \frac{n_A}{n_B} G_B = \frac{275}{225} \times 19 = 23.2 \text{ secs.}$$

(vi) Total cycle length $= G_A + A_A + R_A = G_A + A_A + G_B + A_B$

$$= 23.2 + 4 + 19 + 3 = 49.2 \text{ secs.}$$

Hence adopt cycle length of 50 secs.

The additional period of 50 − 49.2 = 0.8 secs. is distributed to green timings in proportion to approach traffic volume.

$$G_A = 23.2 + 0.44 = 23.64 \text{ secs.}$$

$$G_B = 19.0 + 0.36 = 19.36 \text{ secs.}$$

$$R_A = G_B + A_B = 19.36 + 3.0 = 22.36 \text{ secs.}$$

$$R_B = G_A + A_A = 23.64 + 4.0 = 27.64 \text{ secs.}$$

Design of pedestrian signal

Do not Walk (DW) period of pedestrian signal at road A (PS_A) is red period of traffic signal at B.

For PS_A, $DW_A = R_B = 27.64$ secs.

For PS_B, $DW_B = R_A = 22.36$ secs.

Pedestrian clearance intervals (CI) are of 15 and 10 secs. respectively, for roads A and B for crossing from (ii) above. The walk time (W) is calculated from total cycle length.

For PS_A, $W_A = 50 - (27.64 + 15) = 7.36$ secs.

For PS_B, $W_B = 50 - (22.36 + 10) = 17.64$ secs.

Details of design timings are tabulated in Table 5.9. Alternatively a phase diagram may be drawn, as shown in Fig. 5.31.

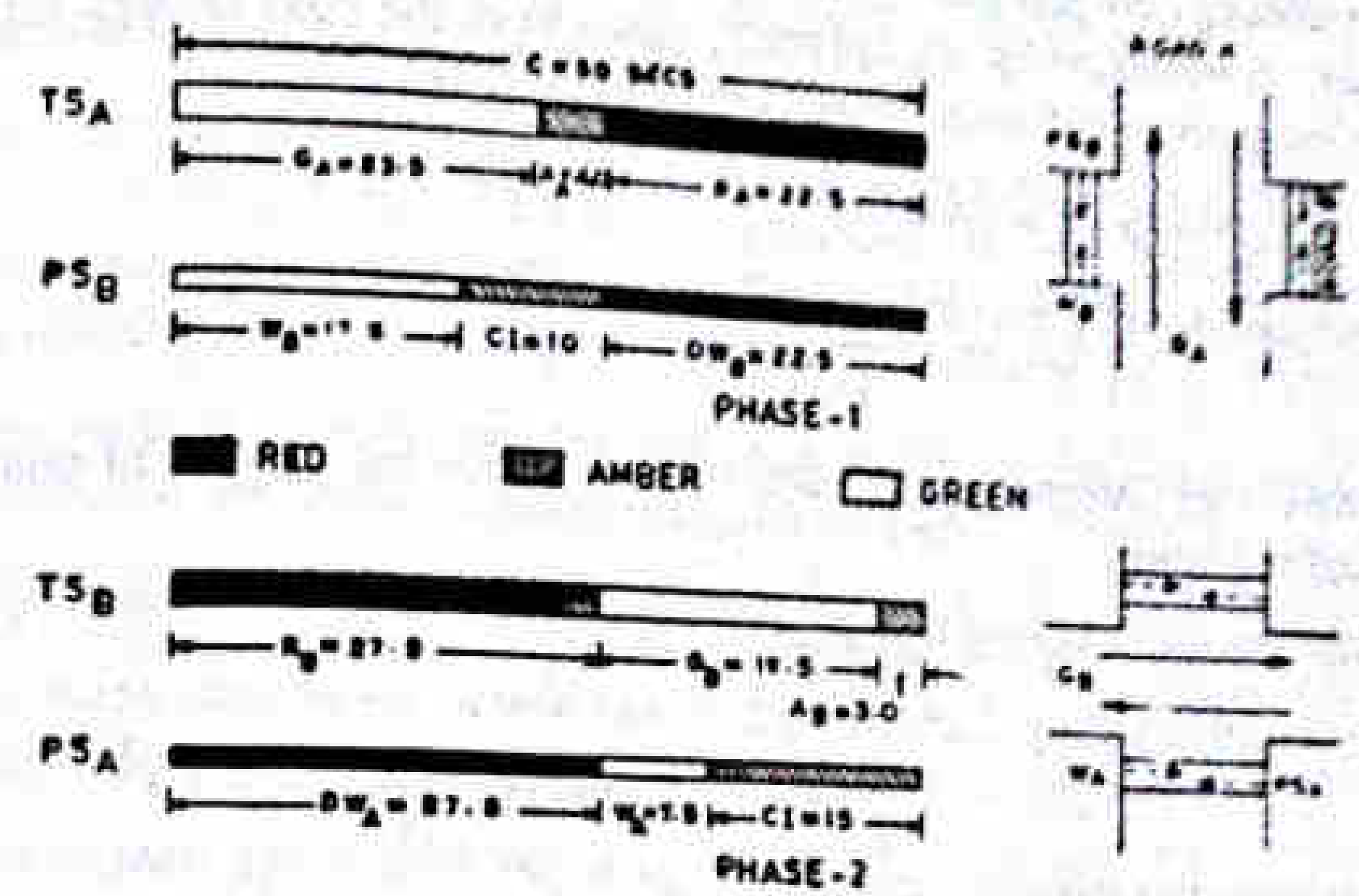


Fig. 5.31 Traffic and Pedestrian Signal Settings

Table 5.9 Details of Design Timings (Ex. 5.13)

Interval	Road A: Vehicle TS_A	Road A: Pedestrian PS_B	Road B: Vehicle TS_B	Road B: Pedestrian PS_A	Time Interval (%)	Actual time, interval seconds: Pedestrian	Actual time, interval seconds: Traffic
1	G23.64	W 17.64	R 27.64	DW 27.64	35	$DW_A = 27.5$ { W_B 17.5, 6.0	$G_A = 23.5$ } $R_B = 27.5$
2		CI 10.00			12		
3	A 4.00				8	CI_B } $4.0A_A$	
4	R 22.36	DW 22.36	G19.36	W 7.36	15	$DW_B = 22.5$ { W_A 17.5, 12.0	$G_B = 19.5$ } $R_A = 22.5$
5				CI 15.00	24		
6			A 3.00		6	CI_A } $3.0A_B$	
Total	50.0		50.0	50.0	100%	50.0 secs	50.0

Traffic signal
R-red (stop)
A-amber (clearance)
G-green (go)

Pedestrian signal
W-walk
Cl-clearance for pedestrians (may be indicated by flickering of walk signal W)
DW-do not walk

Note : Amber period of a short interval however be provided in the traffic signal in between red and green periods for getting ready to cross. This period, is a part of the red period itself.

(3) Webster's method

In this method, the optimum signal cycle C_o corresponding to least total delay to the vehicles at the signalized intersection has been worked out. This is a rational approach. The field work consists of finding (i) the saturation flow S per unit time on each approach of the water section and (ii) the normal flow q on each approach during the design hour. Based on the higher value of normal flow, the ratio $y_1 = q_1/S_1$ and $y_2 = q_2/S_2$ are determined on the approach roads 1 and 2. In the case of mixed traffic, it is necessary to convert all the normal flow and saturation flow values in terms of suitable PCU values which should be determined separately.

The saturation flow is to be obtained from careful field studies by noting the number of vehicles in the stream of compact flow during the green phases, and the corresponding time intervals precisely. In the absence of data the approximate value of saturation flow is estimated assuming 160 pcu per 0.3 metre width of the approach. The normal flow of the traffic is also determined on the approach roads from the field studies for the design period (during the peak or off-peak hours as the case may be).

The optimum signal cycle is given by :

$$C_o = \frac{1.5L+5}{1-Y} \quad (5.24)$$

where L = total lost time per cycle, secs. = 2n + R (n is the number of phase and R is all red-time)

$$Y = y_1 + y_2$$

Then,

$$G_1 = \frac{y_1}{Y}(C_o - L) \text{ and } G_2 = \frac{y_2}{Y}(C_o - L) \quad (5.25)$$

Similar procedure is followed when there are more number of signal phases.

Example 5.15

The average normal flow of traffic on cross roads A and B during design period are 400 and 250 pcu per hour; the saturations flow values on these roads are estimated as 1250 and 1000 pcu per hour respectively. The all-red time required for pedestrian crossing is 12 secs. Design two phase traffic signal by Webster's method.

Solution

$$y_a = \frac{q_a}{S_a} = \frac{400}{1250} = 0.32$$

$$y_b = \frac{q_b}{S_b} = \frac{250}{1000} = 0.25$$

$$Y = y_a + y_b = 0.32 + 0.25 = 0.57$$

$$L = 2n + R = 2 \times 2 + 12 = 16 \text{ secs.}$$

$$C_o = \frac{1.5L+5}{1-Y} = \frac{1.5\times16+5}{1-0.57}$$

$$= \frac{29}{0.43} = 67.4 \text{ say, } 67.5 \text{ secs.}$$

$$G_a = \frac{y_a}{Y}(C_o - L) = \frac{0.32}{0.57}(67.5 - 16) = 29 \text{ secs.}$$

$$G_b = \frac{y_b}{Y}(C_o - L) = \frac{0.25}{0.57}(67.5 - 16) = 22.5 \text{ secs.}$$

All-red time for pedestrian crossing = 12 secs.

Providing Amber times of 2.0 secs. each for clearance, total cycle time = 29 + 22.5 + 12 + 4 = 67.5 secs.

Note : Sketch a phase diagram as shown in Fig. 5.29 or 5.31 or prepare a table to show the signal phases as given in Table 5.9

(4) Design Method as per IRC Guideline

(i) The pedestrian green time required for the major and minor roads are calculated based on walking speed of 1.2 m/sec. and initial walking time of 7.0 secs. These are the minimum green time required for the vehicular traffic on the minor and major roads respectively.

(ii) The green time required for the vehicular traffic on the major road is increased in proportion to the traffic on the two approach roads.

(iii) The cycle time is calculated after allowing amber time of 2.0 secs. each.

Note : The steps mentioned above are similar to the Approximate Method explained under Method (2) and Example 5.14.

(iv) The minimum green time required for clearing vehicles arriving during a cycle is determined for each lane of the approach road assuming that the first vehicle will take 6.0 secs. And the subsequent vehicles (PCU) of the queue will be cleared at a rate of 2.0 secs. The minimum green time required for the vehicular traffic on any of the approaches is limited to 16 secs.

(v) The optimum signal cycle time is calculated using Webster's formula (explained in method 3). The saturation flow values may be assumed as 1850, 1890, 1950, 2250, 2550 and 2990 PCU per hour for the approach roadway widths (kerb to median or centre line) of 3.0, 3.5, 4.0, 4.5, 5.0 and 5.5 m; for widths above 5.5 m, the saturation flow may be assumed as 525 PCU per hour per metre width. The lost time is calculated from the amber time, inter-green time and the initial delay of 4.0 secs. for the first vehicle, on each leg.

(vi) The signal cycle time and the phases may be revised keeping in view the green time required for clearing the vehicles and the optimum cycle length determined it steps (iv) and (v) above.

The design method is illustrated in Example 5.16.

Example 5.16

At a right angled intersection of two roads, Road 1 has four lanes with a total width of 12.0 m and Road 2 has two lanes with a total width of 6.6 m. The volume of traffic approaching the intersection during design hour are 900 and 743 PCU/hour on the two approaches of Road 1 and 278 and 180 PCU/hour on the two approaches of Road 2. Design the signal timings as per IRC guidelines.

Solution

Design traffic on Road 1 = higher of the two approach volume per lane = 900/2 = 450 PCU/hr.

Design traffic on road 2 = 278 PCU/hr

(i) Pedestrian green time for Road 1 $= \frac{12.0}{1.2} + 7.0 = 17$ secs.

Pedestrian green time for Road 2 $= \frac{6.6}{1.2} + 7.0 = 12.5$ secs.

Green time for vehicles on Road 2, G_2 = 17.0 secs.

(ii) Green time for Road 1, $G_1 = 17 \times \frac{450}{278} = 27.5$ secs.

(iii) Adding 2.0 secs. each towards clearance amber and 2.0 secs. inter-green period for each phase, total cycle time required $= (2 + 17 + 2) + (2 + 27.5 + 2) = 52.2$ secs.

Signal cycle time may be conveniently set in multiples of five secs. and so the cycle time = 55 secs.

The extra 2.5 secs. per cycle may be apportioned to the green times of Roads 1 and 2 as 1.5 and 1.0 secs. and so $G_1 = 27.5 + 1.5 = 29.0$ secs. and $G_2 = 17.0 + 1.0 = 18.0$ secs.

(iv) Vehicle arrivals per lane cycle on Road 1

$$450/55 = 8.2 \text{ PCU}$$

Minimum green time for clearing vehicles on Road 1

$$= 6 + (8.2 - 1.0)\,2 = 20.4 \text{ secs.}$$

Vehicle arrivals per cycle on Road 2

$$= 278/55 = 5.1 \text{ PCU}$$

Minimum green time for clearing vehicles on Road 2

$$= 6 + (5.1 - 1.0)\,2 = 14.2 \text{ secs.}$$

As the green time provided for the two roads by pedestrian crossing criteria in (iii) above are higher than these values, the above design values are alright.

(v) Lost time per cycle = (amber time + inter-green time + time lost for initial delay of first vehicle) for two phases = $(2 + 2 + 4) \times 2 = 16$ sec.

Saturation flow for Road 1 $= 525 \times 6 = 3150$ PCU/hr

Saturation flow for Road 2 $= 1850 + \frac{40 \times 3}{5} = 1874$ PCU/hr

$$y_1 = \frac{900}{3150} = 0.286 \text{ and } y_2 = \frac{278}{1874} = 0.148$$

$$Y = 0.286 + 0.148 = 0.434$$

Optimum cycle time

$$C_o = \frac{1.5\,L + 5}{1 - Y} = \frac{1.5 \times 16 + 5}{1 - 0.434} = 51.2 \text{ secs.}$$

Therefore the cycle time of 55 secs. designed earlier is acceptable. The details of the signal timings are given below. These may either be shown in the form of phase diagram as in Fig. 5.29 and 5.31 or in a tabular form as in Table 5.9.

Road	Green	Amber	Red	Cycle
Road 1	29	2	(22 + 2)	55
Road 2	18	2	(33 + 2)	55

Road marking

Road or traffic markings are made of lines, patterns, words, symbols or reflectors on the pavement, kerb, sides of islands or on the fixed objects within or near the roadway. Traffic markings may be called special signs intended to control, warn, guide or regulate the traffic. The markings are made using paints in contrast with colour and brightness of the pavement or other back ground. Light reflecting paints are also commonly used for traffic marking. In order to ensure that the markings are seen by the road users, the longitudinal lines should be atleast 10 cm thick and the transverse lines should be made in such a way that they are visible at sufficient distance in advance to give road users adequate time to respond.

The various types of markings may be classified as,

(a) Pavement markings

(b) Kerb markings

(c) Object markings and

(d) Reflector unit markings

Pavement Markings

Pavement or carriageway markings may generally be of white paint. Yellow colour markings are used to indicate parking restrictions and for the continuous centre line and barrier line markings. Longitudinal solid lines are used as guiding or regulating lines and are not meant to be crossed by the driver. Transverse solid lines indicate the position of stop lines for vehicular traffic.

Some of the common types of pavement markings are given below :

(a) *Centre Lines* : These are meant to separate the opposing streams of traffic on undivided two-way roads. On rural highway with two or three lanes, single broken lines of width 0.1 m and length 4.5 segments and 7.5 m gaps may be painted on straight stretches of NH and SH, these may be decreased to 3.0 and 6.0 m at horizontal curves and approaches to intersection. On other roads at straights the segments are 3.0 m in length and gaps 6.0 m (which are reduced to 3.0 m at curves and approaches to intersection). On four or six lane undivided roads two solid continuous parallel lines of 0.1 m width with 0.05 to 0.10 m space in between are painted.

On urban roads with less than four traffic lanes the centre line consists of white broken lines of width 0.10 to 0.15 m, length of segment 3.0 m and length of gaps 4.5 m to be reduced to 3.0 m at curves and approaches to intersections. On undivided roads with atleast two traffic lanes for each direction of traffic flow, the centre line marking shall consist of two solid continuous lines.

(b) *Lane Line* : Lines are drawn to designate traffic lanes. These are used to guide the traffic and to properly utilize the carriageway.

(c) *No Passing Zone Markings* : These are marked to indicate that overtaking is not permitted.

(d) *Turn Markings* : These are useful near intersection to designate proper lateral placement of vehicles before turning to the different directions.

(e) *Stop Lines* : These are meant for vehicles to stop near the pedestrian crossing, signalized intersection etc. where the vehicles have to stop and proceed.

(f) *Cross Walk Lines* : The particular places where pedestrian are to cross the pavement are properly marked by the pavement markings. The width of pedestrian crossing may be between 2.0 and 4.0 m depending on local requirements.

(g) *Approach to Obstructions*: These may be indicated by appropriate pavement markings.

(h) *Parking Space Limits*: For proper utilization of parking facility, markings are made.

(i) Border or edge lines indicate carriageway edges of rural roads which have no kerb stones along the edges.

(j) Route direction arrows are marked by one or more arrows to guide effectively the traffic into correct lanes.

(k) Parking space limits on urban roads are marked to promote efficient use of parking spaces in a systematic manner.

(*l*) Bus Stops : The length of kerb which is reserved for buses to stop are marked by continuous yellow line on the kerb indicating 'parking prohibited'. The pavement space meant for bus stop is also marked by the word 'BUS'.

Kerb Markings

These may indicate certain regulations like parking regulations. Also the markings on the kerb and edges of islands with alternate black and white line increase the visibility from a long distance.

Object Markings

Physical obstruction on or near the roadway are hazardous and hence should be properly marked. Typical obstructions are supports for bridge, signs and signals, level crossing gates, traffic islands, narrow bridges, culver head walls etc.

Reflector Unit Markings

Reflector markers are used as hazard markers and guide markers for safe driving during night. Hazard markers reflecting yellow light should be visible from a long distance of about 150 m.

Road Delineators

Road delineators are devices or treatment to outline the roadway or a portion there-of to provide visual assistance to drivers about the alignment of a road ahead, especially at night. Three types of delineators that may be used are Roadway Indicators, Hazard Markers and Object Markers.

Roadway indicators are in the form of guide posts, 0.8 to 1.0 m high and painted by black and white strips with or without reflectors and are intended to delineate the edges of the roadway so as to guide the drivers about the alignment ahead. Hazard markers are approximately 1.2 m high plates on posts, either with three red reflectors or markers with black and yellow strips at 45° towards the side of obstruction, meant to define obstructions or objects close to road. Object markers are circular red reflectors arranged on triangular or rectangular panels and are used to indicate hazard and obstructions within the path of vehicles, like the channelizing island placed close to the intersections.

Traffic islands

Traffic islands are raised areas constructed within the roadway to establish physical channels through which the vehicular traffic may be guided. Traffic islands often serve more than one function.

The traffic islands may be classified based on the function as :

(i) Divisional islands

(ii) Channelizing islands

(iii) Pedestrian loading islands

(iv) Rotary

Divisional islands are intended to separate opposing flow of traffic on a highway with four or more lanes. By thus dividing the highway into two one-way roadways, the head-on collisions are eliminated and in general other accidents are also reduced. The width of the divisional islands should be large if the head light glare is to be reduced during night driving. The kerb should be high enough to prevent vehicles from entering into the islands.

Channelizing islands are used to guide the traffic into proper channel through the intersection area. Channelizing islands are very useful as traffic control devices for intersection at grade, particularly when the area is large. The size and shape of the channelizing islands will very much depend upon the layout and dimensions of the intersections. Considerable professional experience and skill is required for the successful design of channelizing islands. If the islands are not properly designed and placed, there is a possibility of violation of rules be the traffic resulting in greater hazards. The various uses of properly designed channelizing islands are listed below :

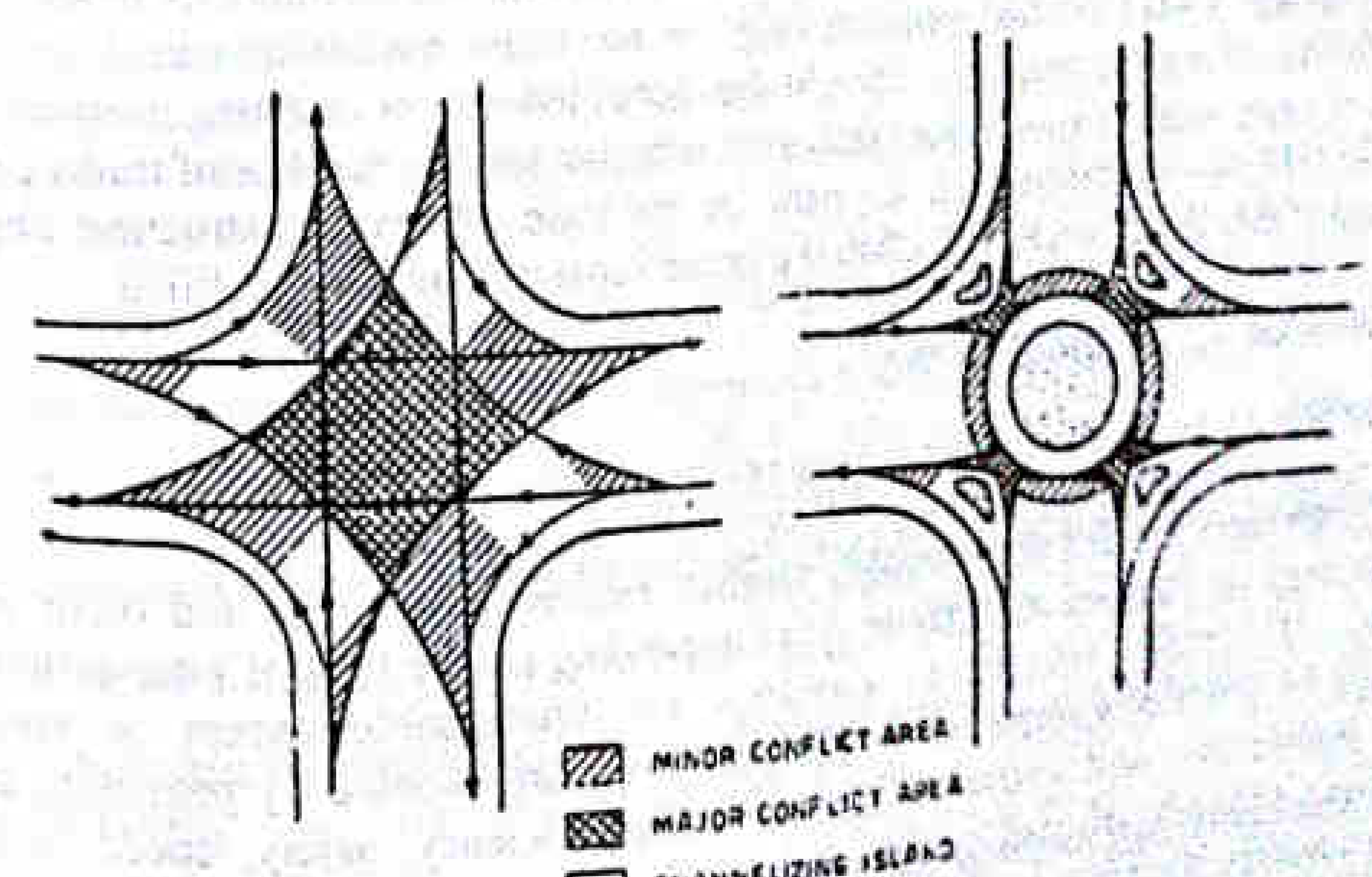
(i) The area of possible conflicts between traffic stream is reduced. This is illustrated in Fig. 5.32. By introducing channelizing islands both the major and minor conflict areas are reduced.

(ii) They establish the desired angles of crossing and merging of traffic streams.

(iii) They are useful when the direction of the flow is to be changed.

(iv) They serve as convenient locations for other traffic control devices.

(v) They serve as refuge islands for pedestrians.



(a) *Area of conflict without channelizing island* (b) *Area of conflict with channelizing islands*

Fig. 5.32 Conflict Areas

The design and functions of *rotary islands*, has been discussed in detail in Art. 5.4.1 under traffic rotary.

Pedestrian loading islands are provided at regular bus stops and similar places for the protection of passengers. A pedestrian island at or near a cross walk to aid and protect pedestrian crossing the carriageway may be termed as pedestrian refuge islands. For crossing multilane highways, pedestrian refuge island after two or three lanes would be desirable. The area in the roadway adjacent to the kerb which is kept reserved for use by stopped bus may be called a bus kerb loading zone.

Rotary island is the large central island of a rotary intersection; this island is much larger than the central island of channelized intersection. The crossing manoeuvre is converted to weaving by providing sufficient weaving length. Further details are given under Rotary Intersection in Art. 5.4.2.

5.3.3 Control of Access on Highways

If effective access control is not affected along a highway facility, ribbon development and encroachments follow, resulting in increase in the number of accidents and considerable reduction in level of service for vehicle operation. The control of access can either be full or partial. Full control of access on highways means that the authority to control the access is exercised to give preference to through traffic by providing access connections with selected public roads only and by prohibiting crossings at grade or direct private drive way connection. When there may be some private drive way connections and some crossings at grade, this is called partial control of access.

Express ways are divided arterial highways for motor traffic with full or partial control of access and generally provided with grade separation at intersection. Arterial highways are primarily meant for through traffic, usually on a continuous route and have partial control of access.

Major corridors of inter-city traffic are increasing in importance and are to be protected from unregulated road side development by exercising limited access control.

Grade separation across highways may be provided at intersections of divided rural highways, if the AADT of fast vehicles only on the cross road within next five years exceeds 5000. Grade separation should be provided across existing railway level crossings, if the product of AADT of fast road vehicles and the number of trains per day exceeds 50,000 within the next five years; in the case of new construction like bye passes, even if this figure exceeds 25,000 the grade separation may be justified.

5.4 DESIGN OF INTERSECTION

5.4.1 General

At the intersection there are through, turning and crossing traffic and these traffic movements may be handled in different ways depending on the type of intersection and its design. Generally intersection problems are unavoidable except in case of expressways or freeway systems where such problems are avoided by providing grade separated intersection and controlled access. The efficiency, safety, speed, cost of operation and capacity of road system very much depend on the intersection design. Pedestrian movements at intersection procedure increased produce hazards and delays.

Intersections may be classified into two broad groups :

(i) *Intersection at grade* : These include all roads which meet at more or less the same level. The traffic manoeuvres like merging, diverging and crossing are involved in the intersections at grade.

(ii) *Grade separated intersection* : The intersecting roads are separated by difference in level, thus eliminating the crossing manoeuvres.

Some of the traffic factors to be considered in intersection design are relative speed and maneuver areas.

Relative speed is an important factor in traffic flow at grade. Relative speed is the vector difference in the velocities of two vehicles in the same flow and is the sum of the speeds of approaching vehicles from opposite direction. It is the speed of convergence of vehicles in separate traffic flows as they approach a point of potential collision. Relative speed is dependent on the absolute speeds of the intersecting vehicles and the angles between them. When the angle of merging is small, the relative speed will also be low. If there is a collision between two vehicles at small angle at about the same speed or at low relative speed, the impact would be much less than when vehicle collide at high relative speed. As the relative speed increases, the judgement of drivers regarding time and distance is likely to be more inaccurate, thus increasing the possibility and severity of accidents. Thus in intersection design care has to be taken to keep the relative speed low.

Manoeuvre areas are those areas where, in actual manoeuvre, there is a potential collision and also those like channels of approach and departure where the manoeuvre is influenced. *Elemental manouvre areas* are those formed by only two single one-way lanes of flows when they diverge, merge or cross, these being the simplest of these manoeuvres. But in *multiple manoeuvre areas* where more than two one-lane one-way flows are present, traffic operations are much more complex and hence are to be avoided in intersection design. The point where the possible path of two vehicles intersect is called conflict points and the area containing all possible conflict points is the conflict area. In intersection design the conflict area, especially the major conflict area where more than one vehicle is subjected to conflict simultaneously, should be minimum.

5.4.2 Intersection at Grade

All road intersections which meet at about the same level allowing traffic manoeuvres like merging, diverging, crossing, and weaving are called intersections at grade. These intersections may be further classified as unchannelized, channelized and rotary intersections.

The basic requirements of intersection at grade are :

(i) At the intersection the area of conflict should be as small as possible.

(ii) The relative speed and particularly the angle of approach of vehicle should be small.

(iii) Adequate visibility should be available for vehicles approaching the intersection.

(iv) Sudden change of path should be avoided.

(v) Geometric features like turning radius and width of pavement should be adequately provided.

(vi) Proper signs should be provided on the road approaching intersection to warn the drivers.

(vii) Good lighting at night is desirable.

(viii) If the number of pedestrians and cyclists are large, separate provision should be made for their safe passage in intersections with high volume of fast moving traffic.

The various forms of intersections are shown in Fig. 5.33.

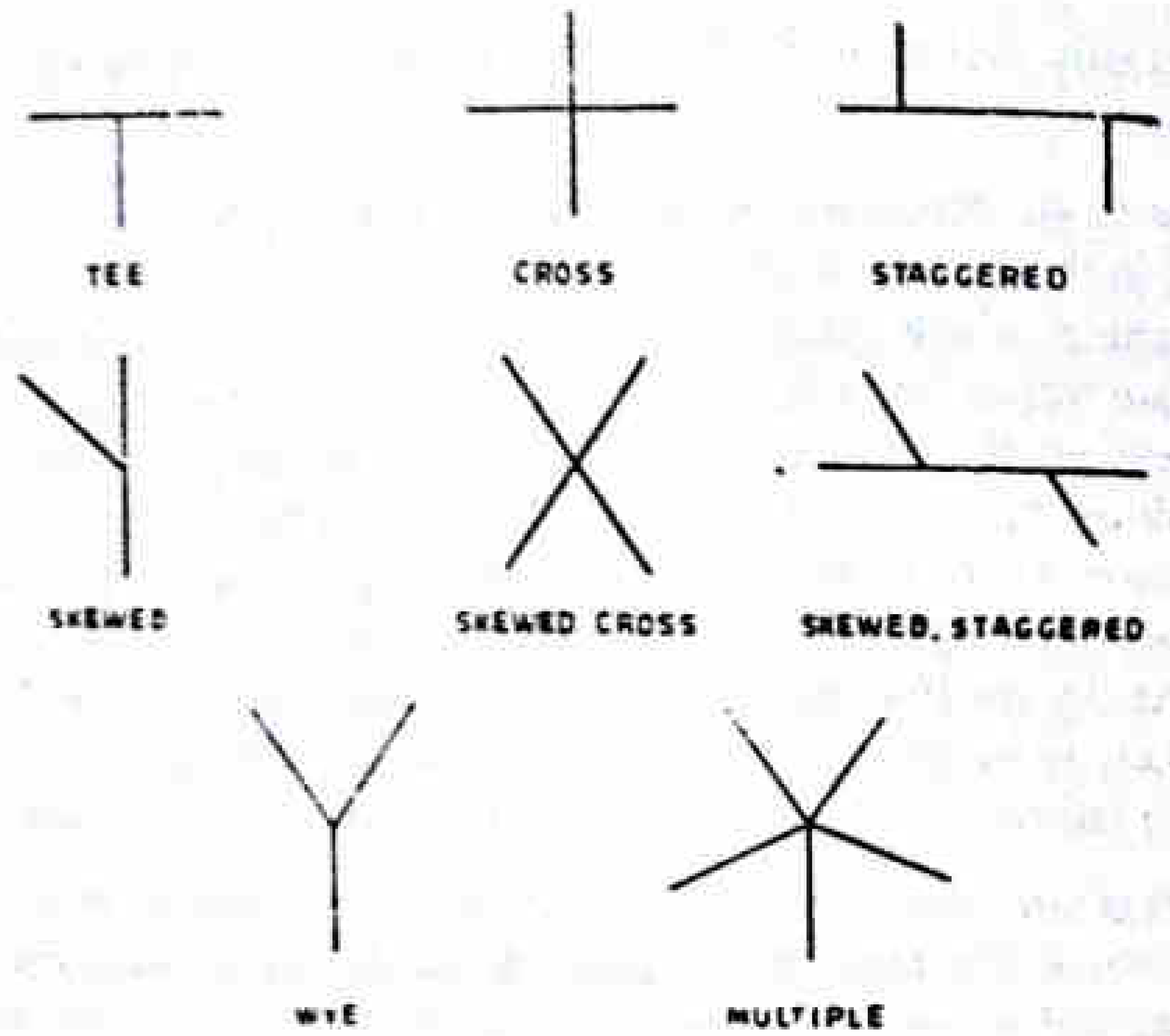


Fig. 5.33 Forms of Intersections

Unchannelized intersections

The intersection area is paved and there is absolutely no restriction to vehicles to use any part of intersection area. Hence the unchannelized (all-paved) intersections are the lowest class of intersection, easiest in the design, easiest in the design; but most complex in traffic operations resulting in maximum conflict area and more number of accidents, unless controlled by traffic signals or police. When no additional pavement width for turning movement is provided, it is called plain intersection. But when the pavement is widened at the intersection area, by a traffic lane or more, it is known as *flared* intersection. These have been illustrated in Fig. 5.34 alongwith common types of unchannelized intersections. The arrows indicate the path of traffic flow, turning, crossing and through movements. It may be seen that the conflict area is quite large as path of turning vehicles are not restricted or controlled. One of the crossing vehicles will have to stop while the other proceeds.

Channelized Intersections

Channelized intersection is achieved by introducing islands into the intersectional area, thus reducing the total conflict area available in the unchannelized intersection. The radius of the entrance and exit curves and the area are suitably designed to accommodate the channelizing islands of proper size and shape. These islands help to channelized turning traffic, to control their speed and angle of approach and to decrease the conflict area at the intersection. Some of the typical examples of channelized intersection are shown in Fig. 5.35. Channelization may be either partial or complete with *divisional* and *directional* islands and medians. From traffic operation point of view there is a better control on the traffic entering and leaving the intersection and hence channelized intersections are considered superior to the all-paved types. However one of the crossing vehicles will have to stop while the other proceeds.

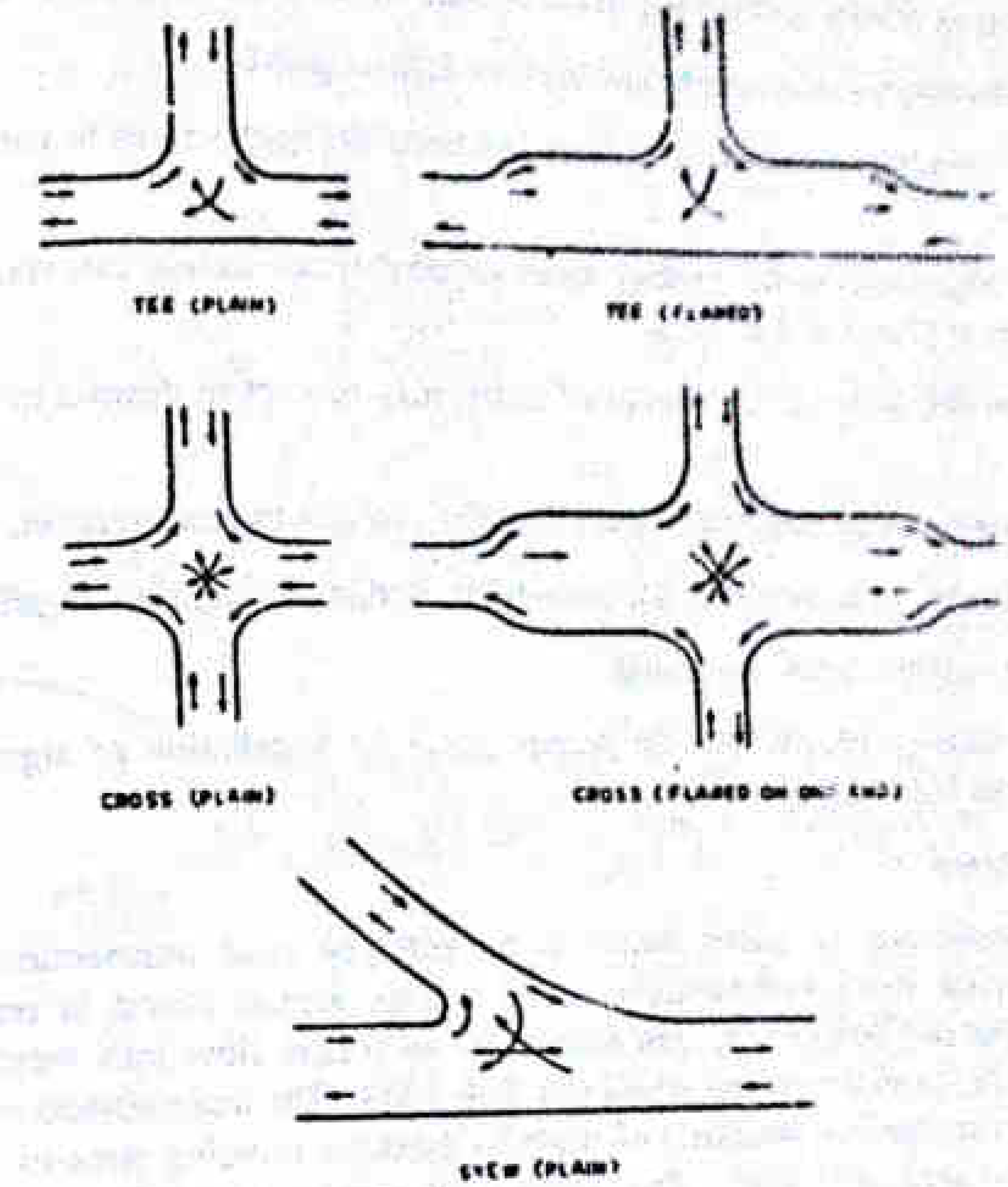


Fig. 5.34 Unchannelized Intersections

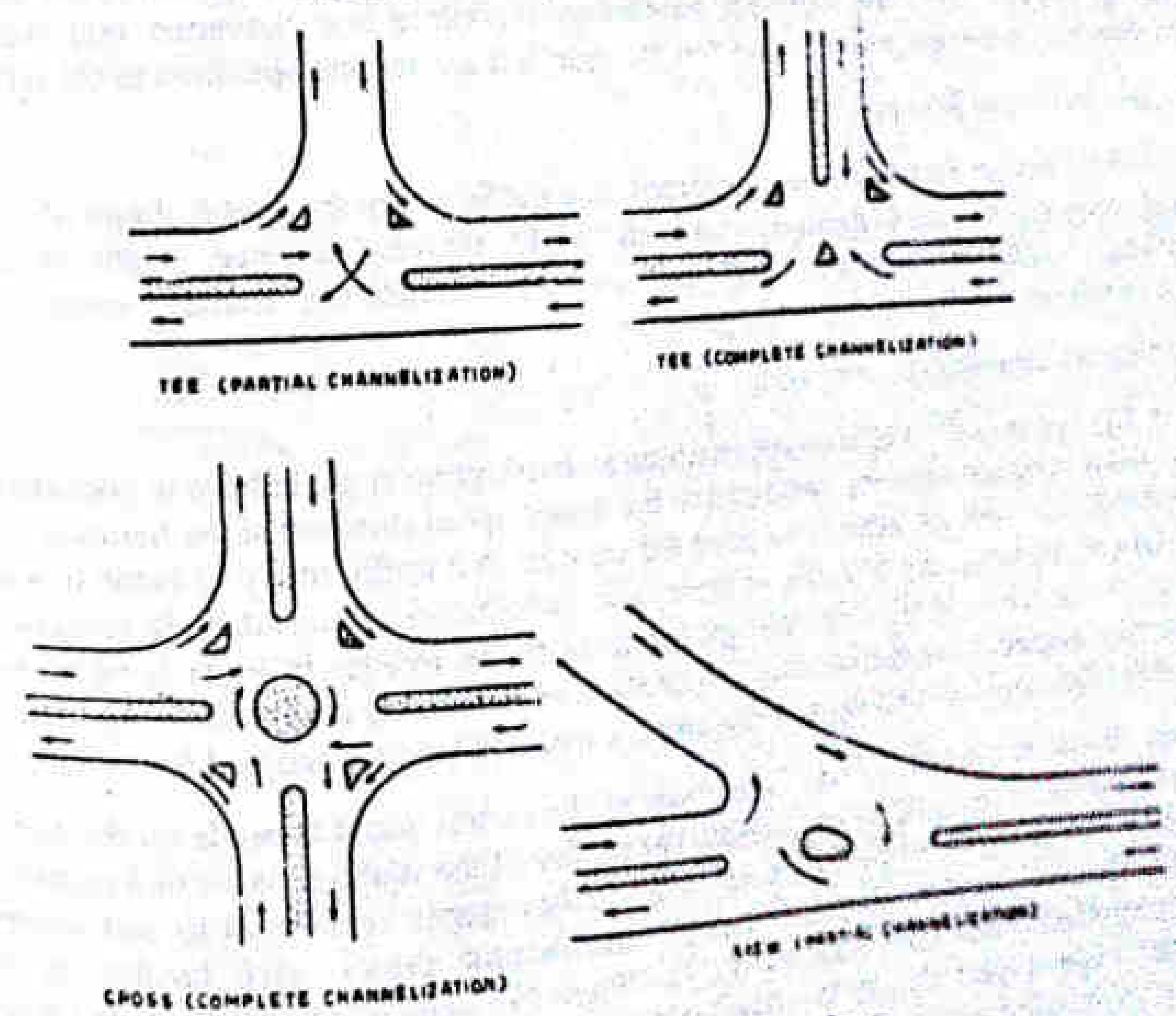


Fig. 5.35 Channelized Intersections

The advantages of channelized intersections may be summed up as follows :

(i) By canalization vehicles can be confined to definite paths.

(ii) Angle of merging streams can be forced to be at flat angles so as to cause minimum disruption.

(iii) Both the major and minor conflict areas within the intersection can considerably be decreased, as shown in Fig. 5.32.

(iv) Angle between intersecting streams of traffic may be kept as desired in a favourable way.

(v) Speed control can be established over vehicles entering the intersection.

(vi) Refuse islands can be provided for pedestrians within the intersection area.

(vii) Points of conflicts can be separated.

(viii) The channelizing islands provide proper place for installation of signs and other traffic control devices.

Rotary intersection

A rotary intersection or traffic rotary is an enlarged road intersection where all converging vehicles are forced to move round a large central island in one direction (clock wise direction) before they can weave out of traffic flow into their respective directions radiating from the central island (see Fig. 5.36). The main objects of providing a rotary are to eliminate the necessity of stopping even for crossing streams of vehicles and to reduce the area of conflict. The crossing of vehicles is avoided by allowing all vehicles to merge into the streams around the rotary and then to diverge out to the desired radiating road. Thus the crossing conflict is eliminated and converted into weaving manoeuvre or a merging operation from the right and a diverging operation to the left.

Design Factors of Rotary

Various design factors to be considered in a traffic rotary are speed, shape of central island, radius of rotary roadway, weaving angle, weaving distance, width of rotary roadway, radius of entrance and exit curves, channelizing islands, camber and superelevation, grade, lighting and signs.

These are briefly explained here.

(i) *Design speed* : Vehicles approaching as intersection at grade have to considerably slow down their speed when compared to the design speed standard of the highway under consideration. Though there is no need for vehicles in a traffic rotary to come to a dead stop before allowing cross traffic to cross, still there has to be considerable reduction in speed. With these in view the design speed for traffic rotaries in India is taken as 40 kmph for rotaries in rural area when one or more of converging roads is/are important. In all other cases and for rotaries in urban areas, a speed 30 kmph is adopted for design.

(ii) *Shape of central island* : The shape of the central island depends on the number and the layout of the intersecting roads. The outline of the island consists of a number of curves of large radii, without corners. The various shapes considered to suit different conditions are circular, elliptical, turbine and tangent shapes, each having its own advantages and limitations (see Fig. 5.37). When two equally important roads cross at roughly right angles i.e., all the four radiating roads placed symmetrically, a circular

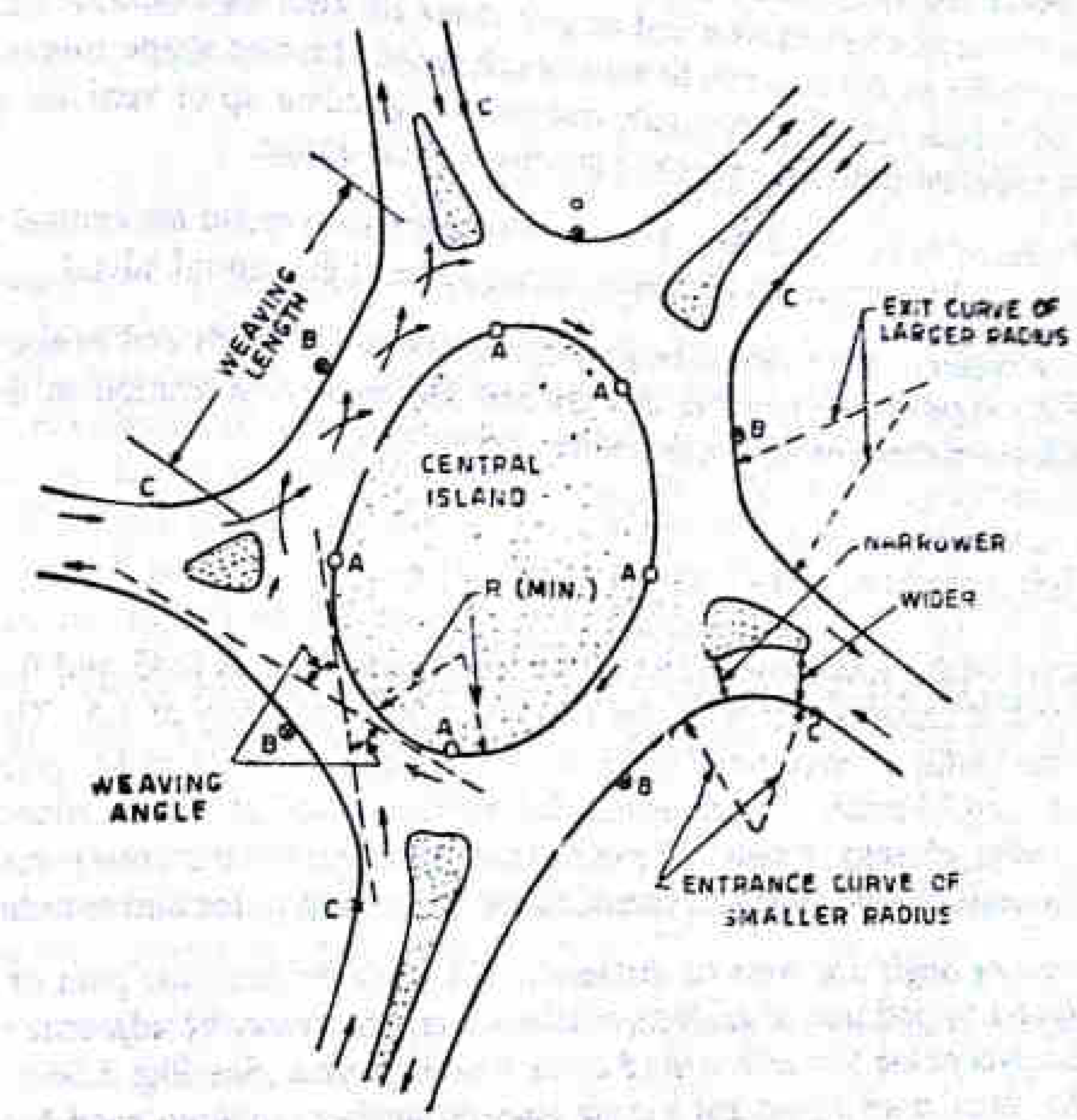


Fig. 5.36 Rotary Intersection

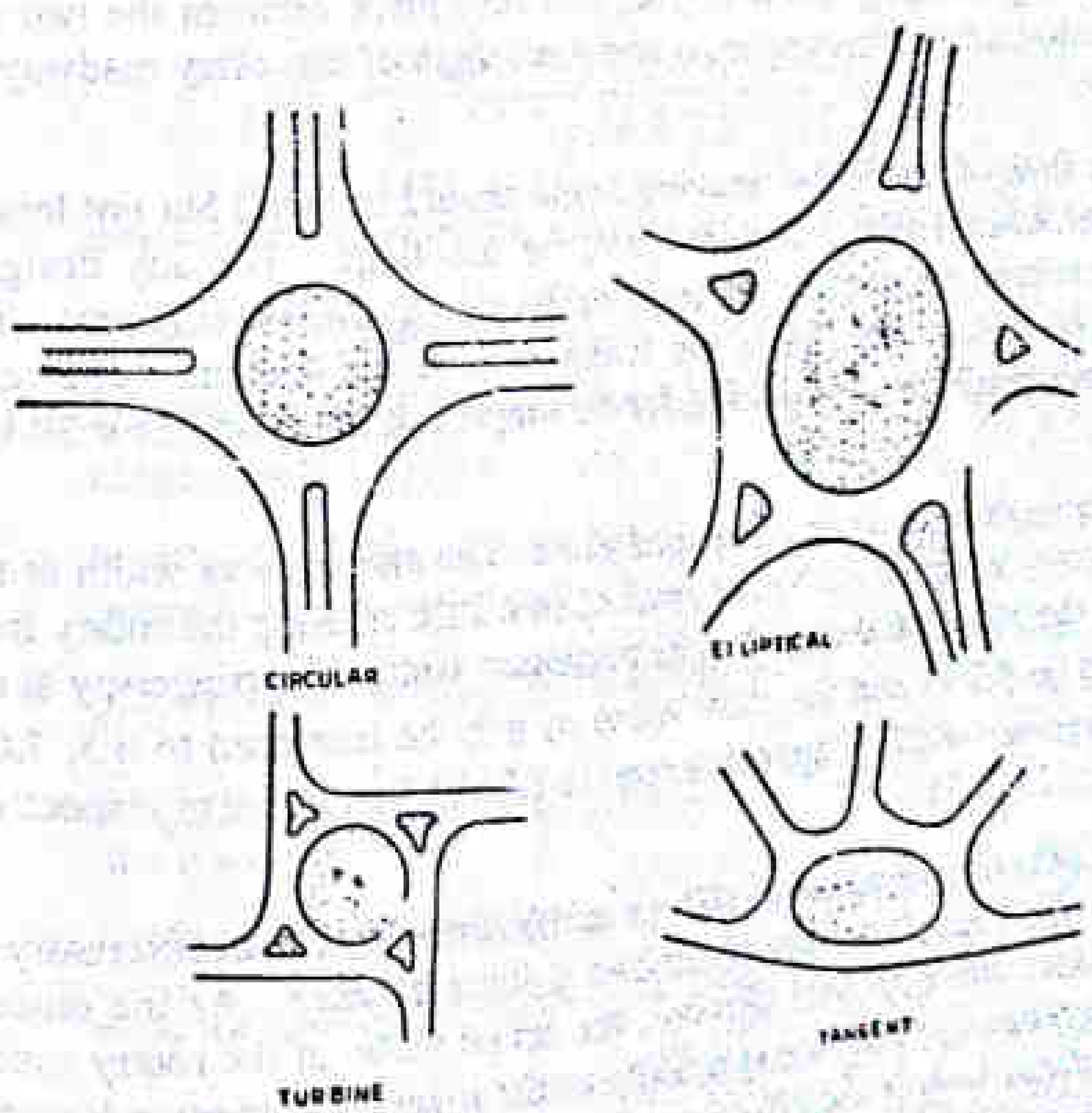


Fig. 5.37 Shapes of Rotary Islands

shape is suitable. The island may be often elongated to accommodate in the layout four or more intersecting roads; and to allow for the greater traffic flow along the direction of elongation. Two much elongation and tangent shape are also not desirable as there is a tendency of traffic in this direction to move much faster. Turbine shape forces reduction in speeds of vehicles entering the rotary and enables speeding up of vehicles going out, however at night, the head light glare is a limitation of the design.

(iii) *Radius of rotary roadway* : The one-way rotary road round the central island has different radii at different points depending on the shape of the central island.

Adequate superelevation cannot be provided on the rotary roads and hence it is safer to neglect the superelevation and to take friction only into consideration in the Eq. 4.8 and 4.9 to arrive at the allowable radius of the curve,

$$R = \frac{V^2}{127\,f}$$

The values of the design coefficient of friction 'f' are taken as 0.43 and 0.47 for the speeds 40 and 30 kmph respectively, after allowing a factor of safety of 1.5. The IRC has suggested the radius of entry curve to be 20 to 35 m and 15 to 25 m for rotary design speeds of 40 and 30 kmph. The recommended minimum radii of central island are 1.33 times the radius of entry curves. Though these radii are for the rotary roadway, in practice it is convenient to design the central island to conform to the above radii.

(iv) *Weaving angle and weaving distance* : The angle between the path of a vehicle entering the rotary and that of another vehicle leaving the rotary at adjacent road, thus crossing the path of the former is termed as the weaving angle (See Fig. 5.36). Vehicles entering the rotary from a road and leaving towards another radiating road have to first merge into the one-way traffic flow in the rotary roadway around the central island and then weave out to diverge from this flow to the required road outlet. The weaving operation including merging and diverging can take place between the two channelizing islands of the adjacent intersecting legs, and this length of the rotary roadway is known as *weaving length.*

For smooth flow of traffic the weaving angle should be small but not less than 15° as the diameter of central island required will be too large. For any design speed the freedom of movement on a rotary depends on the size of the weaving area. The weaving length should be atleast four times the width of weaving section. The recommended value of weaving length are 45 to 90 m for 40 kmph and 30 to 60 m for 30 kmph design speeds.

(v) *Width of carriageway at entry and exit* : The carriageway width at the entrance and exit of a rotary is governed by the amount of traffic entering the rotary from the road or that leaving the rotary to the road. The minimum width of carriageway at the entrance and exit should be 5.0 m and the entry width e_1 may be increased to 6.5, 7.0 and 8.0 m when the carriageway width of approach road is 7.0, 10.5 and 14.0 m respectively and the radius at entry is 25 to 35 m.

(vi) *Width of rotary roadway* : All the traffic entering the traffic rotary have to go round the one-way rotary roadway for atleast a short distance. As the outer kerb lines follow the entrance and exit-sides of roads, the actual width of the rotary roadway varies from section to section. The minimum width of the roadway between edge of the central island and adjoining kerb is the effective width of the rotary roadway or of the weaving section and this by and large determines the capacity of the rotary.

The width of non-weaving section e_2 of the rotary should be equal to the widest single entry to the rotary and should generally be less than the width of weaving section. The width of weaving section W of the rotary should be one traffic lane wider than the mean width of the entry and non-weaving section i.e.;

$$W = \left[\frac{(e_1+e_2)}{2}+3.5\right] m \qquad (5.26)$$

(vii) *Entrance and exit curves* : The curve traced by the inner rear wheel of vehicles determines the radius and shapes to which the kerb line is to be set. A vehicle entering a rotary has to slow down to the design speed of the rotary and therefore the radius of the entrance curve should be the same as the minimum recommended radius of the central island. For the design speed of 40 kmph the suggested radius at entry curves is 20 to 35 m and for 30 kmph, 15 to 25 m. It has been seen that the buses and trucks can take right angled turn easily at these curves at the design speeds. Where practicable three centred entry curves may be provided instead of simple circular curve.

Vehicles leaving the rotary would accelerate to the speed of the radiating roads and hence the exit curves should be of a larger radius than entry curves; one and a half to two times radius of entry is considered reasonable.

The normal pavement width at entrance and exit should be equivalent to two lanes in order to prevent clustering of mixed traffic at the approaches. Extra widening has to be provided at the entrance and exit curve.

The pavement width at entrance curve will be higher than at exit curve as the radius of the former is less than the latter.

(viii) Capacity of the rotary : The practical capacity of the rotary is dependent on the minimum capacity of the individual weaving section. The capacity is calculated from the formula :

$$Q_P = \frac{280\,W\,(1+e/W)(1-p/3)}{(1+W/L)} \qquad (5.27)$$

where Q_P = practical capacity of the weaving section of a rotary in pcu per hour.

W = width of weaving section (6 to 18 m)

e = average width of entry e_1 and width of non-weaving section e_2 for the range e/W = 0.4 to 1.0

L = length of weaving section between the ends of channelizing islands in metre for the range of W/L = 0.12 to 0.4

p = proportion of weaving traffic given by

$$p = \frac{b+c}{a+b+c+d} \text{ in the range 0.4 to 1.0}$$

a = left turning traffic moving along left extreme lane

d = right turning traffic moving along right extreme lane

b = crossing/weaving traffic turning towards right while entering the rotary

c = crossing/weaving traffic turning towards left while leaving the rotary

Some corrections have been suggested in the calculated capacity values depending on the entry, exit and internal angles and the pedestrian traffic in the rotary intersection. The IRC has recommended the following PCU values for finding the capacity of the rotary :

Cars, light commercial vehicles and three wheelers	=	1.0
Buses, medium and heavy commercial vehicles	=	2.8
Motor cycles, scooters	=	0.75
Pedal cycles	=	0.50
Animal drawn vehicles	=	4 to 6

(ix) *Channelizing islands* : Channelizing islands should be provided at the entrance and exit of the rotary to prevent undesirable weaving, and turning and to reduce area of conflict. Further these channelizing islands help in forcing the vehicle to reduce their speed to the design speed of the rotary and to serve as convenient place for erecting traffic signs and as a pedestrian refuge. The shape and size of channelizing island is governed by the radius of the rotary the radii of the entrance and exit curves and the angles and layout of the radial road and rotary. The channelizing islands are generally provided with kerbs 15 to 21 cm high.

(x) *Camber and superelevation* : A vehicle passing along a rotary traverses a reverse curve while changing from one-way path of roadway to the exit of the radial road. Hence the cross slope of the rotary roadway at the point of change in direction should be minimum. The inward slope of the cross slope or camber serves as superelevation for the traffic going around the central island, though design of the curve has been made assuming no superelevation. The outer slope of the camber helps the vehicles turning left towards the exit curve to the radiating road.

(xi) *Sight distance, grade* : The sight distance in the rotary should be as large as possible and in no case less than the safe stopping distance for the design speed. The minimum sight distance should be 45 and 30 m for design speeds of 40 and 30 kmph respectively. It is preferable to locate a rotary on level ground. It may also be located on the area which is on a single plane, with the slope not exceeding 1 in 50 with the horizontal.

(xii) *Lighting* : The minimum lighting required is one each on the edge of central island facing each radiating road. (Points A in Fig. 5.36). Additional lights 'B' may be provided when the central island is larger than 60 m diameter. Light 'C' may also be provided near the entrance curve if the pedestrians are large in number.

(xiii) *Traffic signs* : The standard traffic (warning) signs indicating the presence of rotary intersection should be installed at all approaching roads to give advance information to traffic. At night a red reflector or red light is placed at about one metre above the road level on the nose of each directional island and on the kerb of the central island facing the approaching roads. Vertical black and white strips of width 25 to 30 cm painted on kerb of central island and channelizing islands improve visibility.

(xiv) *Provision for cyclists and pedestrians* : One of the main use of traffic rotary of non-stop and consistent journey is lost, if pedestrians are allowed to enter the rotary intersection or if pedestrian crossings are provided and vehicles are controlled by stop signals. Also the rotary would become a constant problem for traffic control and enforcement. Hence as far as possible pedestrians and even cyclists should be isolated

from the general traffic utilizing the rotary. In India the problem is very typical as rotaries are needed in urban areas where the number of pedestrians and cyclists are also high, making the problem complex. If the number of cyclists are less than 50 per hour, they may be permitted to mix up with the other traffic using the rotary ; if they are more, a separate cycle track to segregate cyclists will be desirable. If there are a large number of pedestrians, separate foot path with guard rails should be provided around the rotary on the outer side to prohibit them from entering the rotary. However, if they are allowed to cross along the pedestrian crossing near the channelising islands, there would be problems of stopping the stream of fast vehicles entering and leaving the rotary. Provision of crossing facilities to pedestrian by *subway* or *over bridge* is possible solution, but the proposal would however be costly.

Conditions when traffic rotary is justified

Construction of a traffic rotary needs large area which may be available in rural areas at reasonable cost. But in India generally the volume of fast moving traffic is very low in rural areas. There are various other points to be considered before the construction of a traffic rotary can be justified.

The American Association of State Highway Officials, now *AASHTO* have suggested that the lowest limit of traffic volume when a traffic rotary is justified is bout 500 vehicles per hour on all intersecting roads put together and the maximum limit beyond which rotary may not efficiently function is about 5000 vehicles per hour. However, if a large proportion of traffic is turning traffic, provision of rotary even outside these limits is justified.

However the IRC suggests that the maximum volume of traffic that a rotary can efficiently handle is 3000 vehicles per hour entering from all the legs of the intersection.

Keeping in view the mixed traffic conditions, it is recommended by the Indian Roads Congress that traffic rotaries may be provided where the intersecting motor traffic is about 50 percent or more of the total traffic on all intersecting roads or where the fast traffic turning right is as least as 30 percent of the total traffic.

Advantages and limitations of traffic rotary

Various Advantages of Rotary

(i) Crossing manoeuvre is converted into weaving or merging and diverging operations. Hence there is no necessity of any of the vehicles, even those which have to go in cross directions, to stop and proceed within a traffic rotary. Thus the journey is more consistent and comfortable when compared with any other intersection at grade.

(ii) All traffic including those turning right or going straight across the rotary have equal opportunity as those turning left.

(iii) The variable cost of operation of automobile is less at a traffic rotary than at a signalized intersection where the vehicles have to stop and proceed. Though the distance to be traversed by vehicles which are to turn to the right or proceed straight across is higher, still the fuel consumed in the process of crossing the rotary intersection is likely to be less. This is because one stop-proceed operation at a signal is likely to consume fuel required for travelling about 275 metre at a uniform speed without stopping.

(iv) There is no necessity of traffic police or signal to control the traffic as the traffic rotary could function by itself as a traffic controlled intersection and is the simplest of all controls. The maintenance cost is hence almost nil.

(v) The possible number of accidents and the severity of accidents are quite low because of low relative speed. Further weaving, merging and diverging manoeuvres are easier and less dangerous operation than crossing. Check on speed of vehicles is automatically enforced by proper design.

(vi) Rotaries can be constructed with advantage when the number of intersecting roads is between four and seven.

(vii) The capacity of the rotary intersection is the highest of all other intersections at grade. The rotary can accommodate a total traffic upto about 3000 vehicles per hour and enable radial streets to carry traffic almost to their full capacity.

Various Limitations of Rotary

(i) Rotary requires comparatively a large area of land and so where space is limited and costly as in built up areas, the total cost may be very high.

(ii) Where pedestrian traffic is large as in urban areas the rotary by itself cannot control the traffic and hence has to be supplemented by traffic police. If the vehicular traffic have to stop to allow pedestrian to cross, the main purpose of rotary is defeated.

(iii) In places where there is mixed traffic and large number of cyclists and pedestrians, the design of rotary becomes too elaborate and operation and control of traffic also become complex.

(iv) Where the angle of intersection of two roads is too acute or when there are more than seven intersecting roads, rotaries are unsuitable.

(v) When the distance between intersections on an important highway is less, rotaries become troublesome.

(vi) Where there are a large number of cycle and animal drawn vehicles, the extra length to be traversed by crossing and right turn traffic is considered troublesome and there is a tendency to violate the traffic regulation of clock wise movement around the central island.

(vii) When the traffic volume is very low as in most of the rural areas of India, construction of a rotary cannot be justified.

5.4.3 Grade Separated Intersections

Grade separated intersection design is the highest form of intersection treatment. This type of intersection causes least delay and hazard to the crossing traffic and in general is much superior to intersections at grade from the point of view of traffic safety and efficient operation.

A highway grade separation is achieved by means of vertical level. Separation of intersecting roads by means of a bridge thus eliminating all crossing conflicts at the intersection. The grade separation may be either by an over bridge or under pass Transform of route at the grade separation is provided by interchange facilities consisting of ramps. Interchange ramps may be classified as direct, semi-direct or indirect as shown in Fig. 5.38. The direct interchange ramp involves diverging to right side and merging from the right. Semi-direct interchange ramp allows diverging to left but merging is from right side. In the indirect method of interchange ramp, a simple diverging to the left and a merging from the left side are involved which are simpler and less hazardous than diverging to the right and merging from right; but the distance to be traversed in indirect interchange is more.

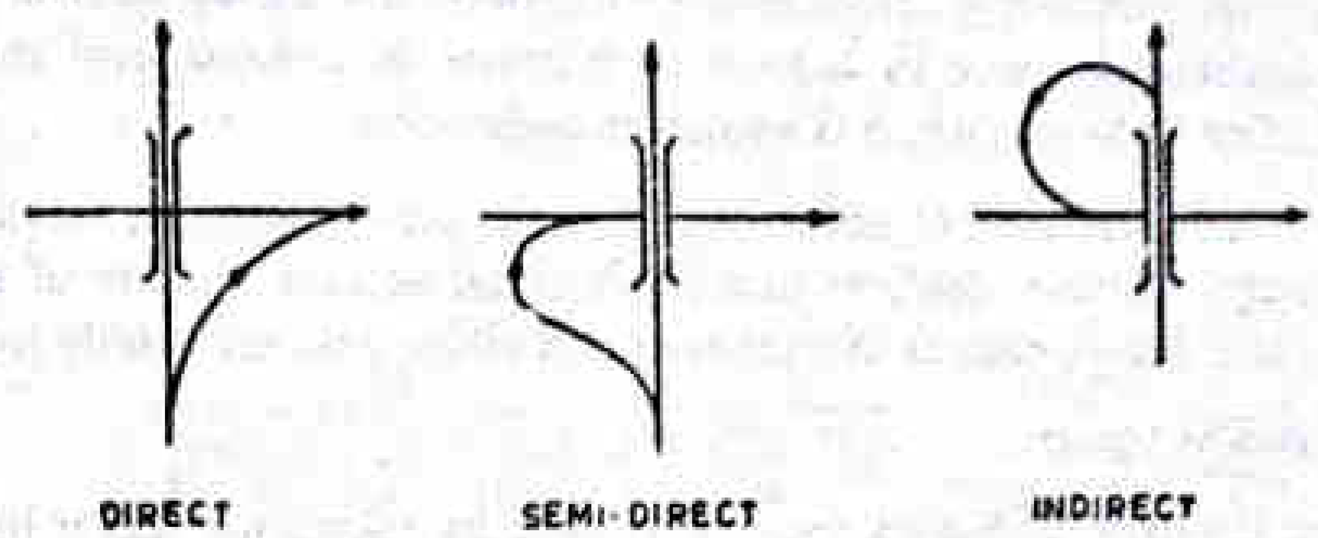


Fig. 5.38 Interchange Ramps

The grade separated intersections have the following advantages and limitations.

Advantages of Grade Separation

(i) Maximum facility is given to the crossing traffic. As the roads are separate, this avoids necessity of stopping and avoids accidents while crossing.

(ii) There is increased safety for turning traffic and by indirect interchange ramp even right turn movement is made quite easy and safe by converting into diverging to left and merging from left.

(iii) There is overall increase in comfort and convenience to the motorists and saving in travel time and vehicle operation cost.

(iv) The capacity of the grade operated intersection can practically approach that of the two cross roads.

(v) Grade separation is an essential part of controlled access highway like expressway and freeway.

(vi) It is possible to adopt grade separation for all likely angles and layout of intersecting roads.

(vii) Stage construction of additional ramps are possible after the grade separation structure between main roads are constructed.

Disadvantages of Grade Separation

(i) It is very costly to provide complete grade separation and interchange facilities.

(ii) Where there is limited right of way like built up or urban area or where the topography is not favourable, construction of grade separation is costly, difficult and undesirable.

(iii) In flat or plain terrain, grade separation may introduce undesirable crests and sags in the vertical alignment.

Grade separation structures

The various types of bridge structures used to separate the grades of the two intersecting highway may be T-beam bridge arch bridge, rigid portal frame type and prestressed concrete bridges. There should be vertical clearance of atleast 4.3 m and if double decked vehicles are anticipated, the clearance should be 5.2 metre. The type of the bridge structure should be selected depending upon the design, construction and other considerations like site conditions and aesthetics.

The grade separated intersections are classified as *over-pass* and *under-pass*. When the major highway is taken above by raising its profile above the general ground level by embankment and an overbridge across another highway, it is called an over-pass. On the contrary if the highway is taken by depressing it below the ground level to cross another road by means of an under-bridge, it is known as under-pass.

The choice of the over-pass or under-pass depends on topography, vertical alignment, drainage, economy, aesthetic features and preferential aspects for one of the highways. The advantages and disadvantages of over-pass and under-pass are briefly listed below.

Advantages of an Over-pass

Troublesome drainage problems may be reduced by taking the major highway above the cross road. For the same type of structure when the wider road is taken above the span of the bridge being small, the cost of the bridge structure will be less. In an over-pass of major highway, there is an aesthetic preference to the main through traffic and less feeling of restriction or confinement when compared with the under-pass. Future expansion or lateral expansion or construction of separate bridge structure for divided highway is possible.

Disadvantages of an Over-pass

In rolling terrain if the major road is to be taken above, the vertical profile will also have rolling grade line. If the major highway is to be taken over by constructing high embankments and by providing steep gradients, the increased grade resistance may cause speed reduction on heavy vehicles. Also there will be restrictions to sight distance unless long vertical curves are provided.

Advantages of an Under-pass

There is a warning to traffic in advance due to the presence of an under pass which can be seen from distance. When the major highway is taken below, it is advantageous to the turning traffic because the traffic from the cross road can accelerate while descending the ramp to the major highway and the traffic from the major highway can decelerate while ascending the ramp to the cross roads. The under-pass may be of advantage when the main highway is taken along the existing grade without alteration of its vertical alignment and cross road is depressed and taken underneath.

Disadvantages of an Under-pass

There may be troublesome drainage problems at the under pass, especially when the ground water level rises high during rainy season and the road at the under-pass is to be depressed as much as 5 m to 7 m below the ground level. It may be necessary even to pump water continuously during the period when water-logging problems exist. At under-pass the over head structure may restrict the vertical sight distance even at the valley curve near the under-pass. There is a feeling of restriction to the traffic at the sides while passing along the under-pass and unless the clearance is sufficiently large, this may affect the capacity at the intersection. There is no possibility of stage construction for the bridge structure at the under-pass.

Interchanges

Grade separated intersection with complete interchange facilities is essential to develop a highway with full control of access. When there is intolerable congestion and accidents at the intersection of two highways carrying very heavy traffic there is no better solution than to provide grade separated intersection.

Some of the types of interchanges are shown in Fig. 5.39. Of all these complete *clover leaf* fulfils all the requirements of turning traffic involving the simplest traffic maneuvers, viz. diverging to the left and merging from the left by providing four indirect ramps.

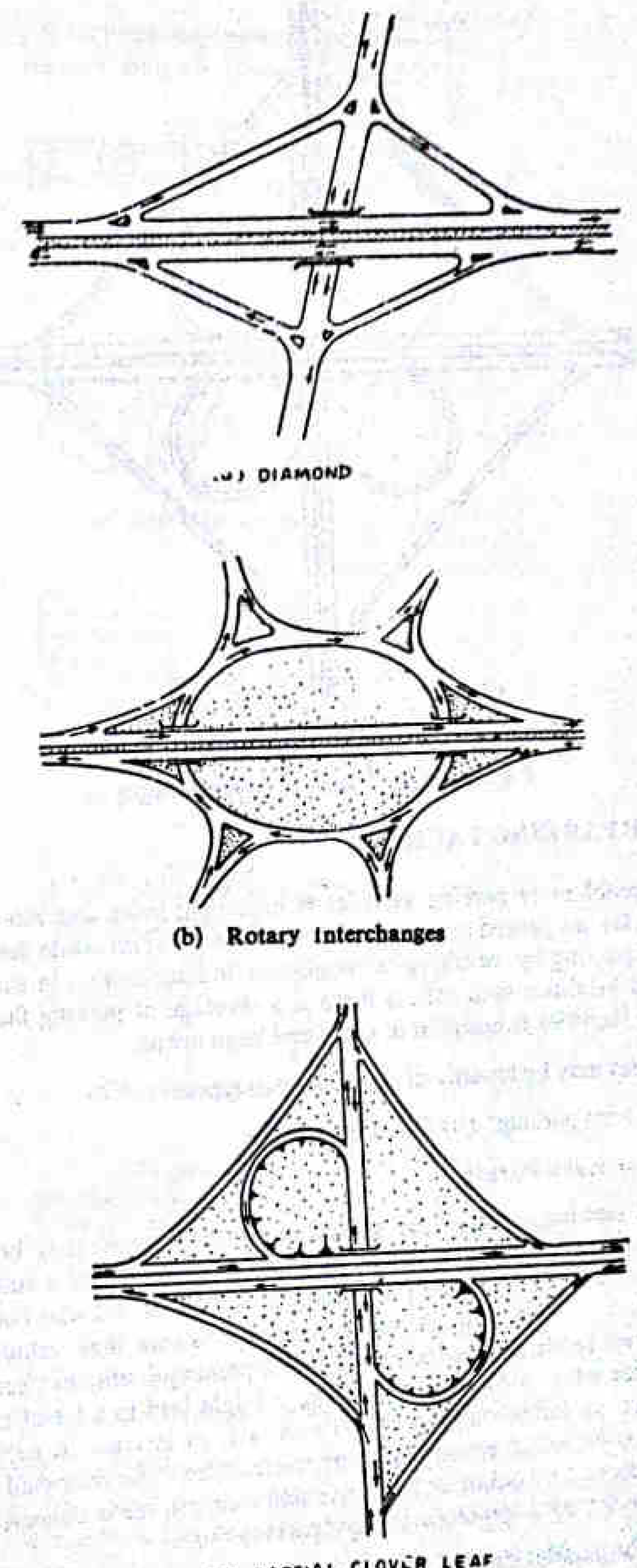


Fig. 5.39 (a) (b) & (c) Types of Interchanges (Contd.)

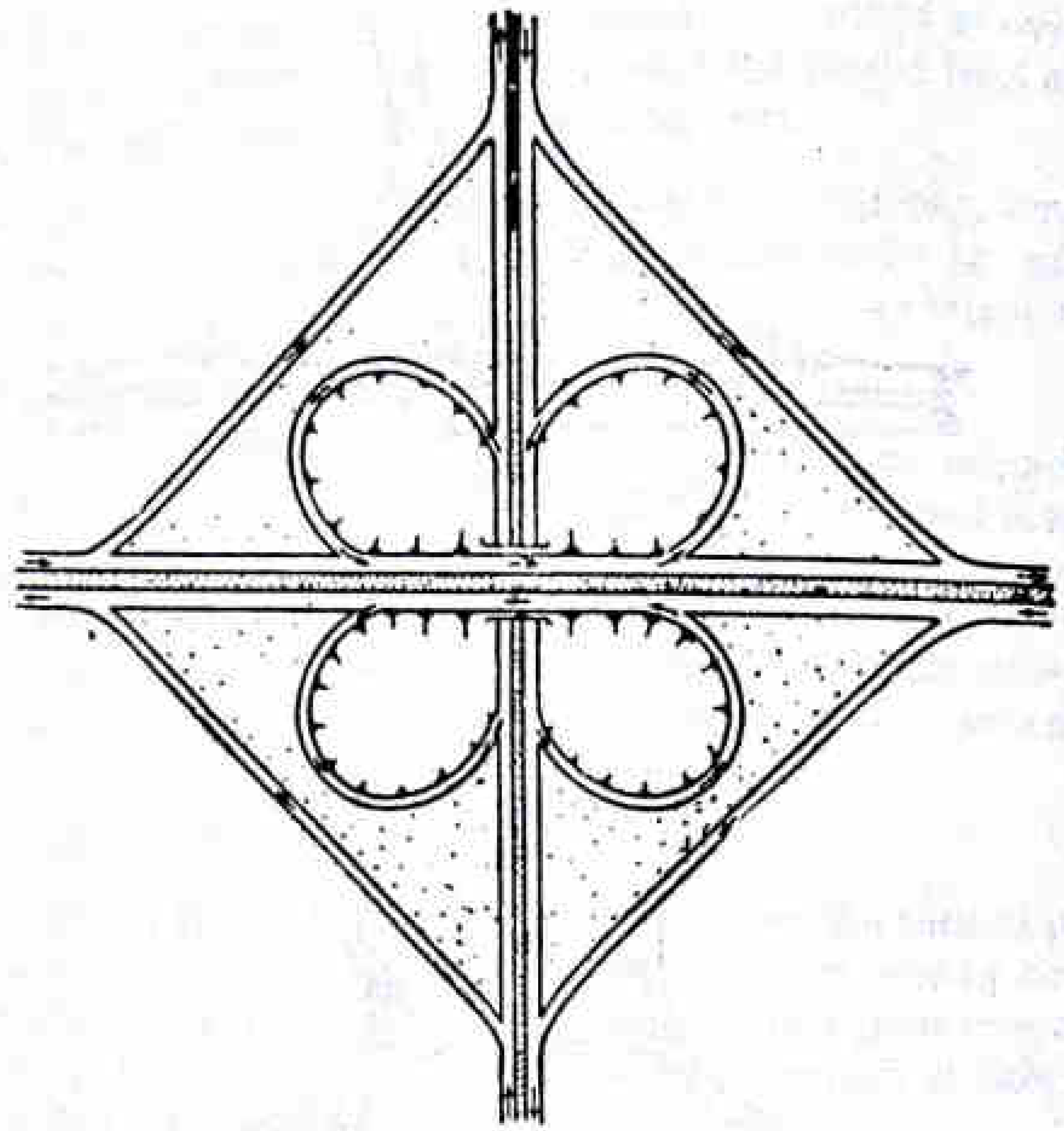


Fig. 5.39 (d) Types of Interchanges

5.5 DESIGN OF PARKING FACILITY

In cities the problem of parking vehicles is becoming more and more acute day by day. When vehicles are parked on the road side, even for a short while there is restriction to other vehicles passing by, resulting in congestion and accidents. In shopping centers, public places and localities with offices there is a shortage of parking facilities. Proper design of parking facilities is essential in cities and large towns.

Parking facilities may be broadly divided into two types :

(i) On-street or kerb parking, (ii) Off-street parking

5.5.1 On-Street or Kerb Parking

In this type of parking, vehicles are parked on the kerb which may be designed for parking. Kerb parking is quite convenient for those who could find a suitable space to park their vehicles near the place they wish to stop; but for others who could not find a parking space it is a problem and often they may have to park their vehicle at a far off place and walk down to the destination. Unless kerb parking facility has been adequately designed in advance while planning a new town, it might lead to a lot of inconvenience and congestion due to decreased road capacity as well as increase in accidents. Kerb parking facility may be either unrestricted or restricted type. The restricted kerb parking may either be controlled by police or by metres and a certain fee is collected from those who park their vehicles for a certain duration of parking time.

Angle parking or parallel parking may be allowed in the kerb parking. See Fig. 5.40. Angle parking may be at angles 30, 60 or 90 degrees. Angle parking accommodates

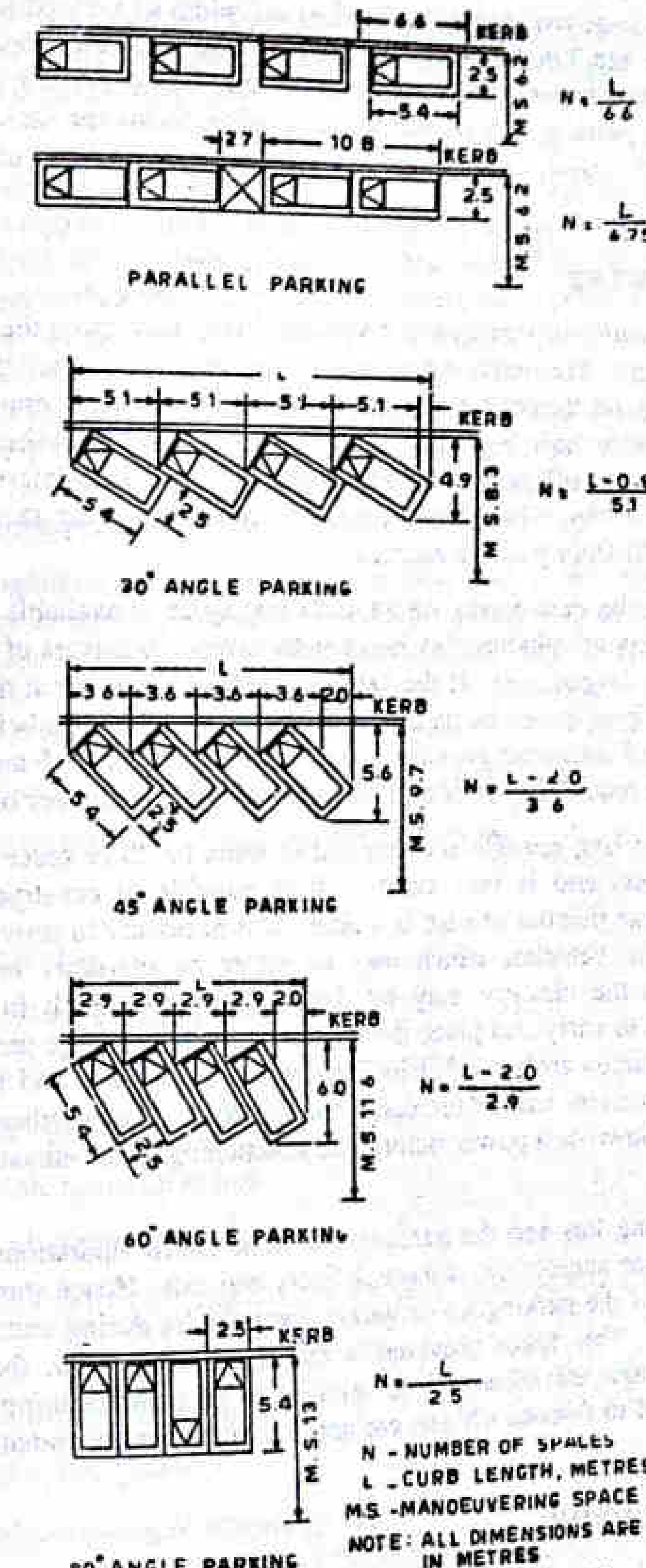


Fig. 5.40 Various Patterns of Kerb Parking

more vehicles per unit length of kerb and maximum vehicles that can be parked is with an angle of 90 degree. The width of road required for parking and unparking manoeuvre also is more with angle parking and it increases with the parking angle up to a maximum at 90° angle. Angle parking is more convenient for the motorists than the parallel parking, but it produces much more obstruction to the through traffic resulting in more accidents than the parallel parking. Out of various angles used in angle parking, 45 degree angle is considered the best from all considerations discussed above.

Parallel parking is generally preferred when the width of kerb parking space and the width of the street are limited. But the parking and unparking operations are more difficult needing a few forward and reverse movements before parking properly or before taking out. Parallel parking may be with equal spacing, facing the same direction or may be two cars placed closely with open intervals between two-car units, as shown in Fig. 5.40.

5.5.2 Off-Street Parking

When parking facility is provided at a separate place away from the kerb, it is known as off-street parking. The main advantage of this method is that there is no undue congestion and delay on the road as in kerb parking. But the main draw back is some of the owners will have to walk a greater distance after parking the vehicle. It is also not possible to provide the off-street parking facility at very close intervals especially in business centers of a city. Two basic types of off-street parking facilities are surface parking lots and multi-floor parking garages.

Parking lots may be convenient where sufficient space is available at comparatively low cost. The parking of vehicles may be done by owners or drivers of the cars and then this is called *self parking system*. If the vehicle is left by the driver at the entrance space and again collected from there, the parking and delivering operations being carried out by attendants, it is called *attendant parking system*. Most important advantage of attendant parking is less space required to store and manoeuvre the same number of cars.

Multi-storeyed parking garages are restored to when the floor space available for the parking garage is less and is very costly. It is possible to construct multi-storeyed garages to park a large number of cars at a time. It is necessary to provide the interfloor travel facility for the vehicles, which may be either by *elevators or by ramps*. In mechanized garages the elevator may be designed to move both in vertical and in horizontal directions to carry and place the vehicle in the appropriate parking stall and to deliver it back. If ramps are provided for driving the vehicles to and from the parking stall, the space requirement will be increased considerably. On the other hand, if there is a mechanical break down or a power failure, the functioning of the elevator system would come to a stand still.

Both in the parking lots and the garages, the basic traffic operations consist of five steps, namely, entrance acceptance, storage delivery and exit. Hence some definite space is required in front of the parking lot or garage for vehicles during entrance acceptance and exit operations. This space provided is called *reservoir area*, the size of which depends on the average rate of arrival of vehicles to be parked during peak hour, the average time required to dispose off one car and the number of attendants employed for storage-operations.

5.6 HIGHWAY LIGHTING

The rate of highway accidents and fatalities that occur during night driving is several times higher in terms of vehicle-kilometre, than that during day driving. One of the various causes of increased accident rate during night may be attributed to poor night visibility. Highway lighting is particularly more important at intersections, bridge site, level crossings and in places where there is restriction of traffic to movements. Lighting on rural roads has not yet become common, evidently due to the cost consideration and less number of pedestrians and other slow traffic using the facility at night. On urban

roads where the density of population is also high, road lighting has other advantages like feeling of security and protection. Thus even though head lights of vehicles may be sufficient for safe night driving, still road lighting may be considered as an added facility to the road users.

During night driving the manner in which objects are visible varies with both the absolute level of brightness and the relative brightness of the road surface and the object. When the brightness of the object is less than that of the background, that is when the object appears darker than the road surface, *discernment* is principally by *silhouette*. If the brightness of the pavement is uniformly increased, discernment by silhouette is enhanced. Hence it is obvious that night visibility on concrete and other light coloured pavements are better than on black top surfaces. A light coloured, rough textured pavement surface that can reflect light back is considered most desirable. Surface that becomes mirror like or shiny when wet (such as smoothened black top road surface) should be avoided as practically no light reflects back from them.

When the brightness of an object is more than that of the immediate background, discernment is by *reverse silhouette*. The objects adjacent to the roadway, projections above the pavement surface such as island or vehicles may be seen by this process of reverse silhouette. When the pavement surface is very dark like black top surface, the object which are relatively brighter in colour are seen by this process.

Thus the various factors that influence night visibility are :

(a) amount and distribution of light flux from the lamps.

(b) size of object.

(c) brightness of object.

(d) brightness of the background.

(e) reflecting characteristics of the pavement surface.

(f) glare on the eyes of the driver, and

(g) time available to see an object.

Design factors of highway lighting

Various factors to be considered in the design of road lighting are :

(i) Lamps

(ii) Luminaire distribution of light

(iii) Spacing of lighting units

(iv) Height and over hang of mounting

(v) Lateral placement

(vi) Lighting layouts

Lamps

The choice of the lamp, its type, size and colour depends on several considerations in addition to distribution of light flux on the pavement surface. It is economical to use the largest lamp size in a luminaire which will provide sufficient uniformity of pavement brightness; but this depends on the spacing of the lamps also. The various types of lamps in use for highway lighting are filament, fluorescent and sodium or mercury vapour lamps. The cheapest amongst these, is the filament lamp. Sodium-vapour lamps are preferred at large intersections.

Luminaire Distribution of Light

To have the best utility of the luminaire or source of light, it is necessary to have proper distribution of light. The distribution should be downward so that high percentage of lamp light is utilized for illuminating the pavement and adjacent area. The light distribution selected should be the one which would produce maximum uniformity of pavement brightness. The distribution from the luminaire should cover the pavement between the kerbs and provide adequate lighting on adjacent area i.e. 3 m to 5 m beyond the pavement edge. The illumination is necessary for traffic signs and other objects on the road.

There are five typical luminaire distributions (see Fig. 5.41) which meet most of the highway lighting requirements.

It is suggested that the average level of illumination on road side may be 20 to 30 lux on important urban roads carrying fast traffic and about 15 lux for other main roads carrying mixed traffic and in arterial roads. In secondary road it may be 4 to 8 lux depending on traffic. However the actual intensity of illumination in most of the existing roads may be lower than the above values.

The Indian Standards Institution recommends an average level of illumination of 30 lux on important roads carrying fast traffic and 15 lux on other main roads, the ratio of minimum to average illumination being 0.4.

Spacing of Lighting Units

The spacing of lighting units is often influenced by the electrical distribution poles, property lines, road layout and type of side features and their illumination. Large lamps with high mountings and wide spacings should be preferred from economy point of view.

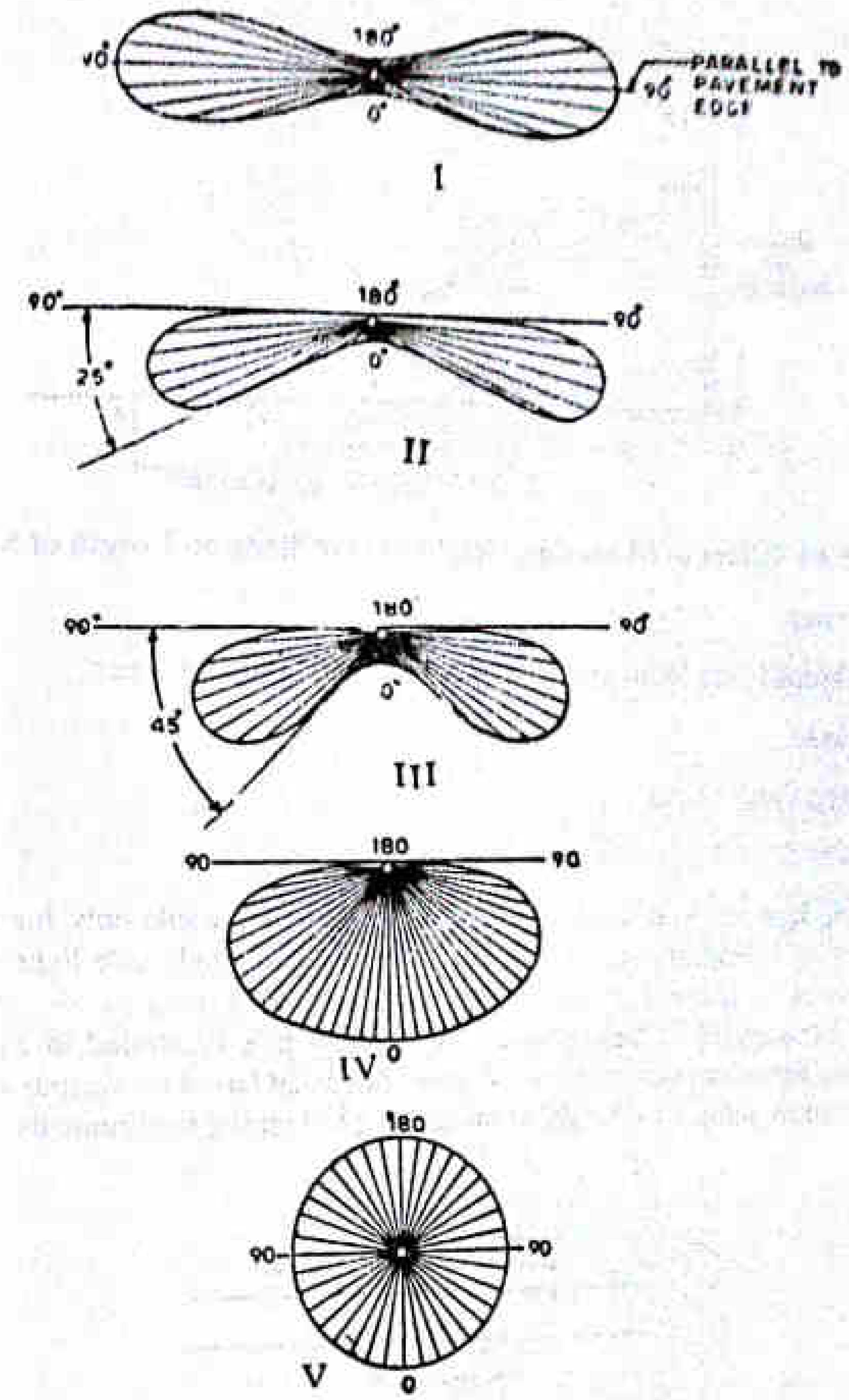
Height and Overhang of Mounting

The distribution of light, shadow and the glare effect from street lamps depend also on the mounting height. The glare on eyes from the mounted lights increases with the power of the lamp directed towards the eye and decreases with increase in height of mounting. Usual mounting heights range from 6 m to 10 m, higher values being preferred where possible at least for important urban roads. The minimum vertical clearance required for electric power lines upto 650 volts has been specified as 6 m above the pavement surface by the Indian Roads Congress.

Over hangs on the lighting poles would keep the poles away from the pavement edges, but still allow the lamp to be held above the kerb or towards the pavement. This enables better distribution of light on the pavement and less glare on eyes of road users. The effect of mounting height and over hang on the length of shadow is shown in Fig. 5.42. It is desirable to have higher mounting heights and necessary overhang projections.

Lateral Placement

The street lighting poles should not be installed close to the pavement edge. If they are too close to the carriageway, free movement of traffic is obstructed, decreasing the capacity of the roadway. Indian Roads Congress has specified the horizontal clearance required for lighting poles as given below :



I Two-way lateral distribution (for narrow roads)

II Narrow asymmetric lateral distribution (for narrow roads)

III Medium width asymmetric lateral distribution (for roads with medium width)

IV Wide asymmetric lateral distribution (for very wide highways)

V Normal symmetrical distribution (for mounting at centre of highways and at intersections)

Fig. 5.41 Types of Luminaire Distribution

(a)	For roads with raised kerbs (as in rural roads)	Minimum 0.3 m and desirable 0.6 m from the edge of raised kerb.
(b)	For roads without raised kerbs (as in rural roads)	Minimum 1.5 m from the edge of the carriageway, subject to minimum of 5.0 m from the centre line of the carriageway.

The clearance specified apply to poles carrying electric power and telecommunication lines also.

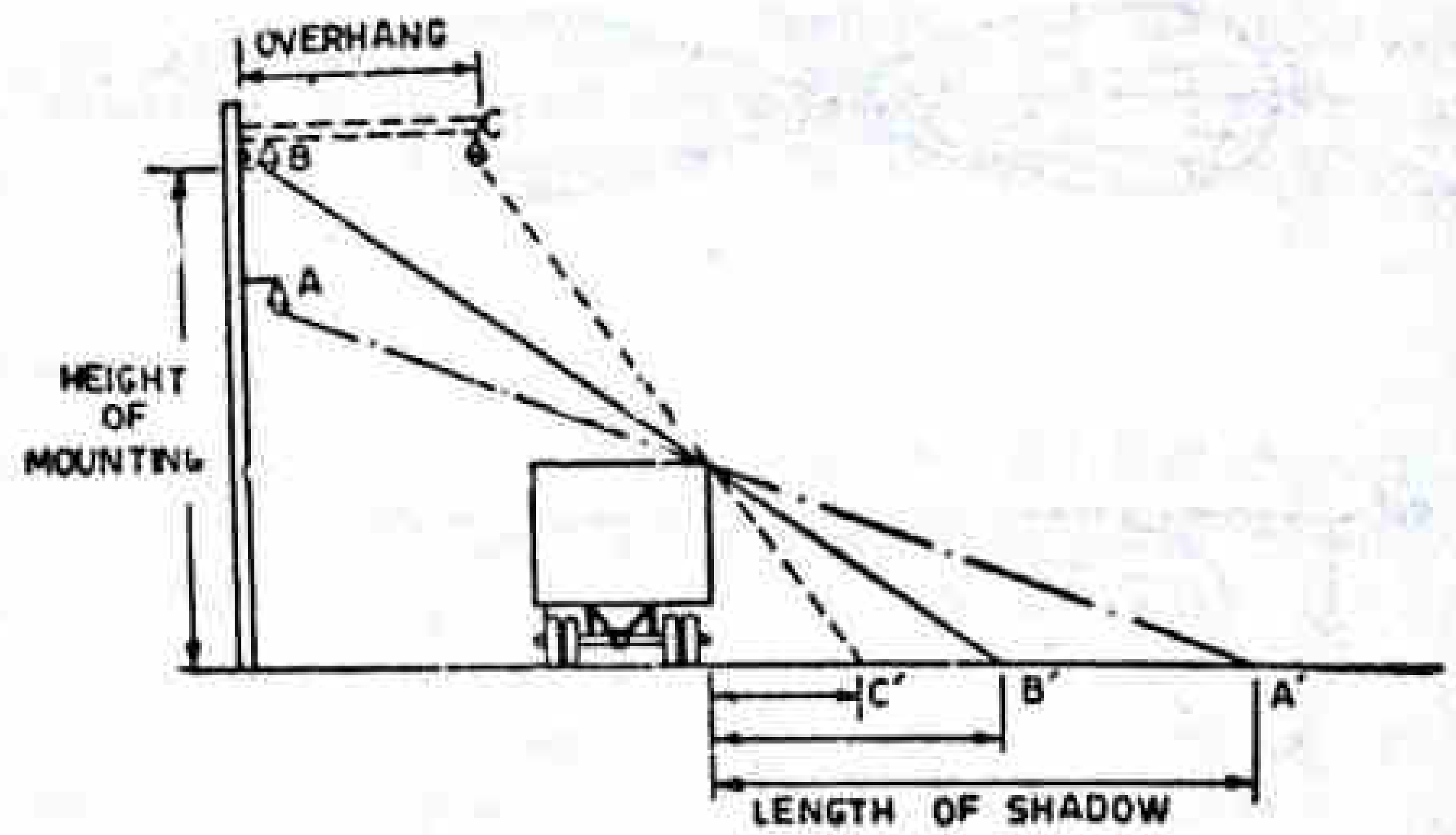


Fig. 5.42 Effect of Mounting Height of Overhang on Length of Shadow

Lighting Layouts

On straight roads the lighting layout may be of the following types :

(a) Single side

(b) Staggered (both sides)

(c) Central

Single side lighting is economical to install; but it is suitable only for narrow roads. Due to cost considerations even on two lane roads often single side lighting is adopted. For wider roads with three or more lanes the *staggered system* or the *central lighting system* may be adopted. These systems of lighting have illustrated in Fig. 5.43. The spacing of the lights in each of these systems is decided based on various considerations including location, lamp size, height of mounting and lighting requirements.

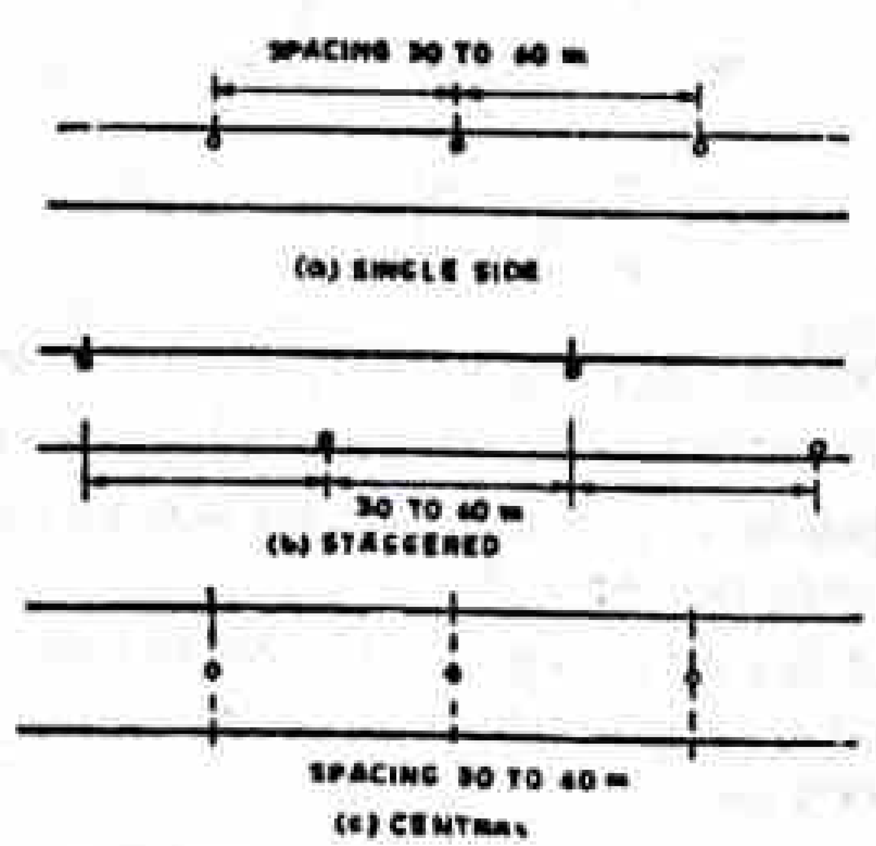


Fig. 5.43 Lighting Layouts

Special care should be taken while locating the lights on curves. Lights are installed at closer spacings on curves than on straights. The lights are located on the outer side of the curves to provide better visibility. The layout of light at horizontal curves is shown in Figure. 5.44. At vertical summit curve lights should be installed at closer intervals near the summit.

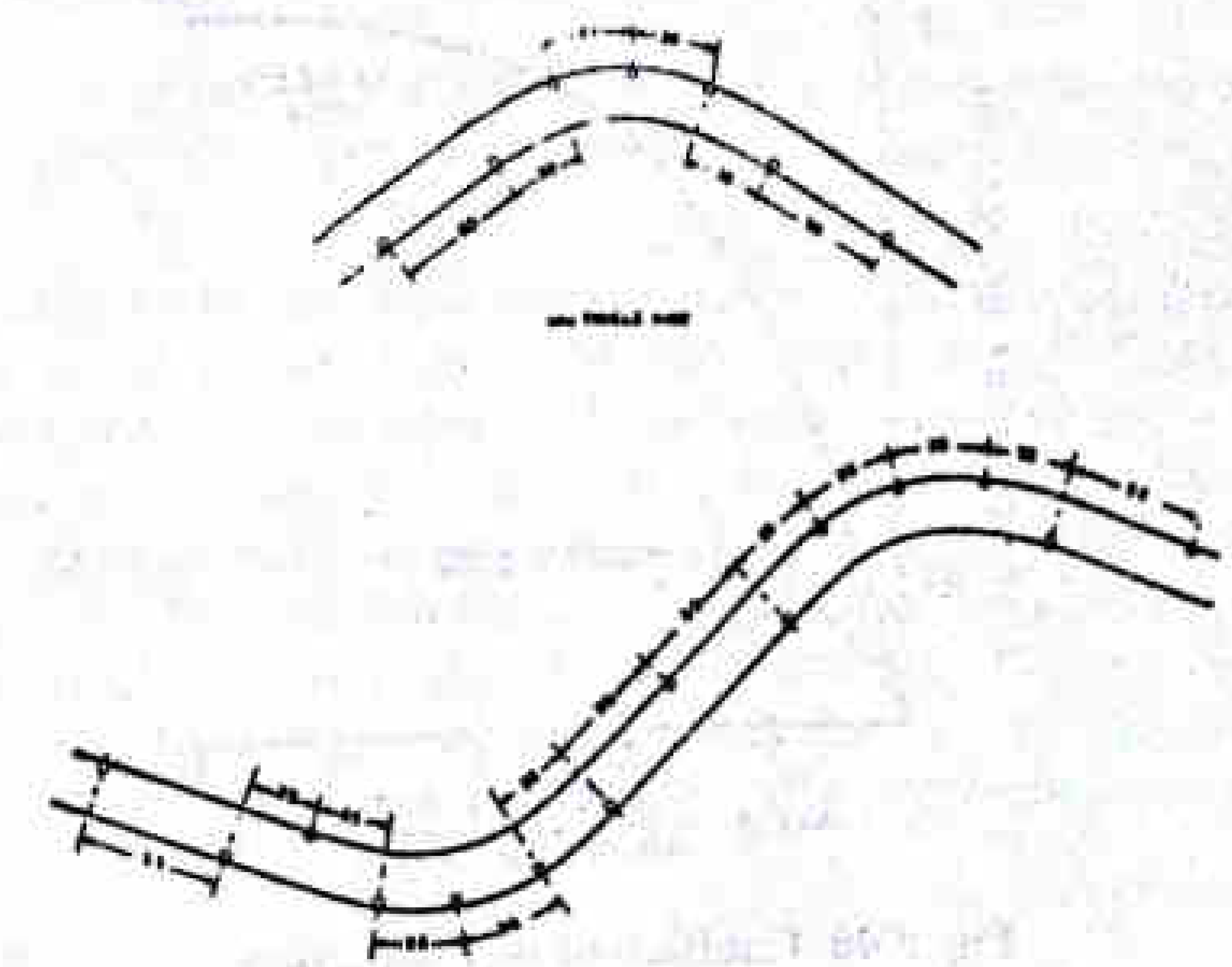


Fig. 5.44 Lighting Layout on Horizontal Curves

At intersections, due to potential conflicts of vehicular and pedestrian traffic, more illumination is required. For simple intersections, in urban area, the illumination should be atleast equal to the sum of illumination values for two roads which form the intersection. See Figure 5.45. A detailed traffic volume and flow study should be made in the cases of compound intersections before deciding the layout of lights. The lighting unit should be located near the pedestrian crossing, channelizing islands and signs. The lighting layout for traffic rotaries has been shown in Fig. 5.46.

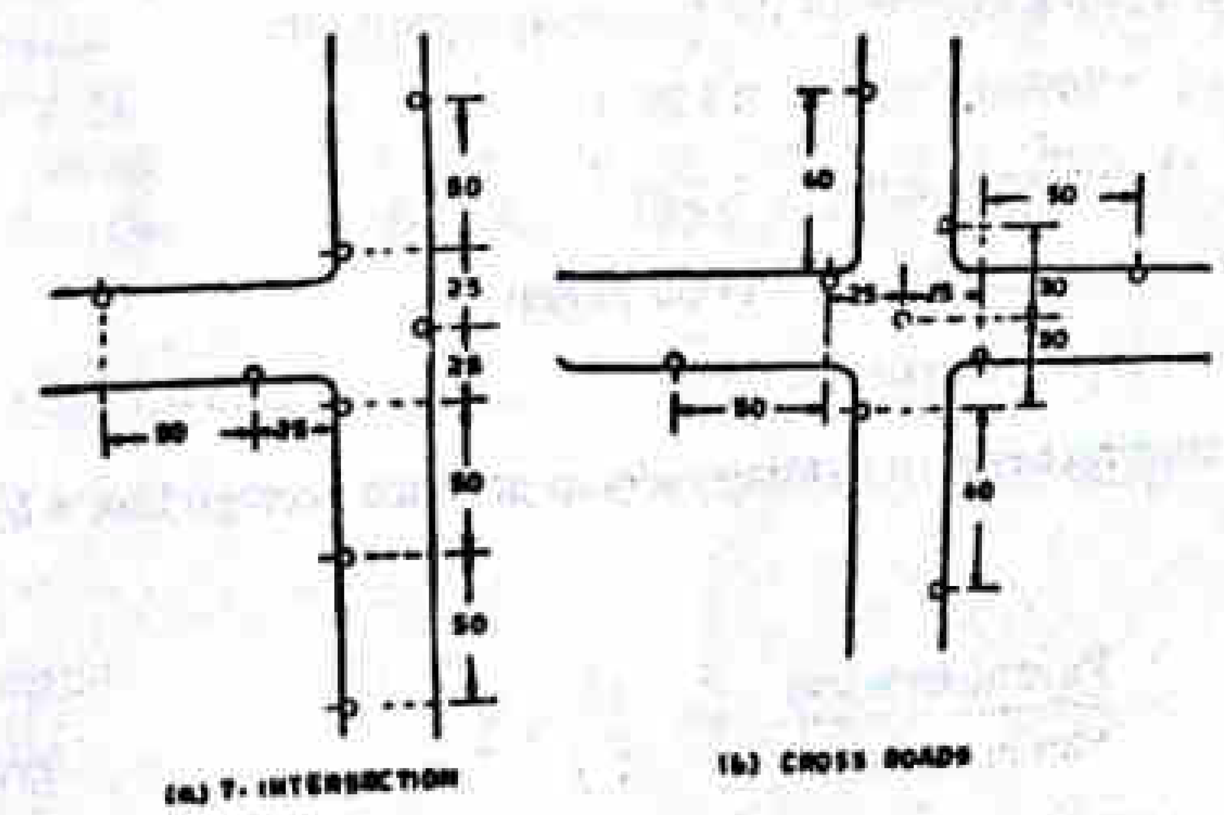


Fig. 5.45 Lighting Layout for Intersections

Design of highway lighting systems

For various types of luminaire distribution, the utilization coefficient charts are available for determination of average lux of intensity over the roadway surface where lamp lumen, mounting height, width of paved area and spacing between lighting poles are known. The typical utilization coefficient chart is given in Fig. 5.46.

The following relationship is used for computations :

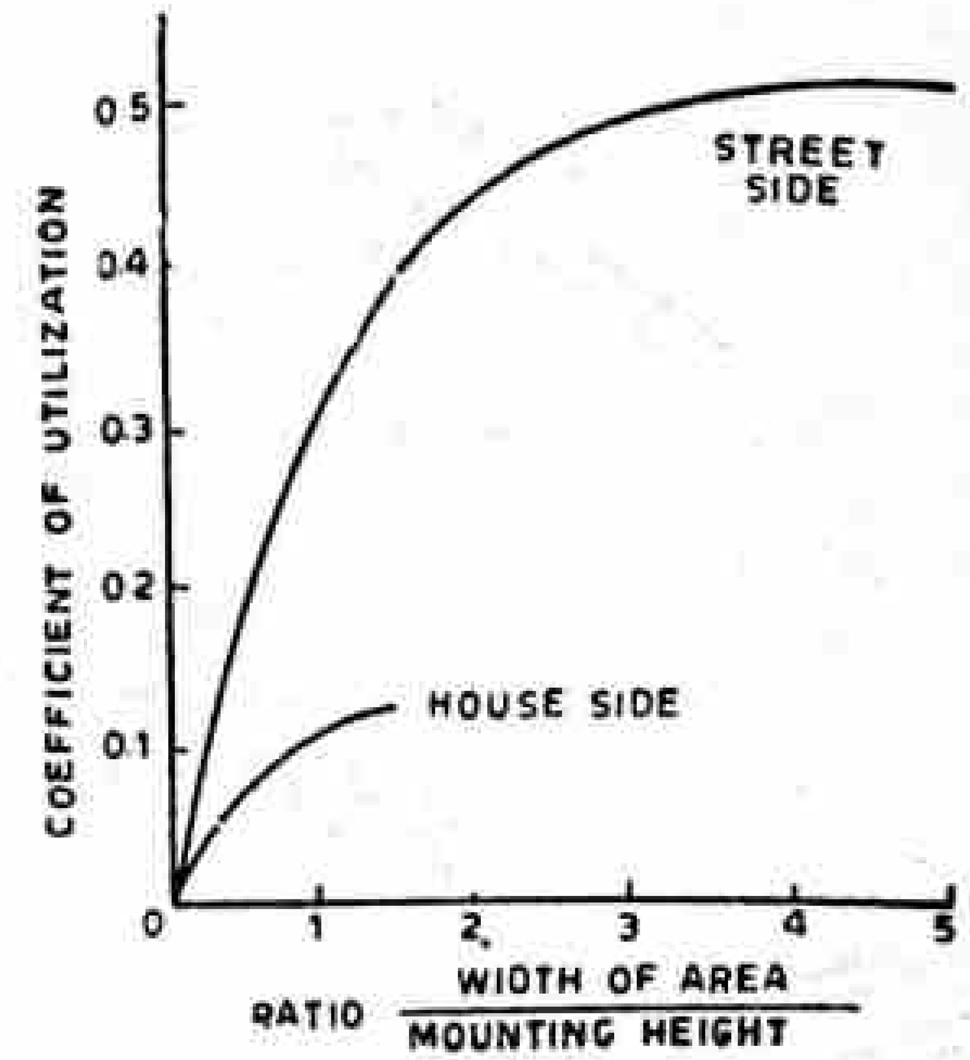


Fig. 5.46 Coefficient of Utilization

$$\text{Spacings} = \frac{\text{Lamp lumen} \times \text{Coefficient of utilization} \times \text{Maintenance factor}}{\text{Average Lux} \times \text{Width of road}}$$

The coefficient of utilization is obtained from the appropriate chart, as in Fig. 5.46. The maintenance factor takes into account the decrease in efficiency of lamp with age and an average value of about 80% may be assumed.

Example 5.16

Design a street lighting system for the following conditions

Street width – 15 m

Mounting height – 7.5 m

Lamp size – 6000 lumen

Luminaire type – II

Calculate the spacing between lighting units to produce average Lux = 6.0

Solution

The ratio $\frac{\text{Pavement width}}{\text{Mounting height}} = \frac{15}{7.5} = 2$

From Fig. 5.46, coefficient of utilization = 0.44

Assume a maintenance factor = 0.8

$$\text{Spacing} = \frac{\text{Lamp lumen} \times \text{Coefficient of utilization} \times \text{Maintenance factor}}{\text{Average Lux} \times \text{Width of road}}$$

$$= \left[\frac{6000 \times 0.44}{6 \times 15} \times 0.8\right] = 23.2$$

5.7 TRAFFIC AND TRANSPORTATION PLANNING

5.7.1 Traffic Planning

Rising trends in growth of population and traffic around cities and the steady growth of national productivity create a continuing demand for improvements in highway facilities.

The problem of traffic accidents and congestion in urban roads is being viewed with grave concern in the recent years. The main causes for this problem are improper planning of road net-work and other roadway facilities and poor traffic planning. Hence traffic functions now occupy a good position in Corporation and Municipalities. The functions and duties of traffic engineering units were initially limited to traffic surveys and control devices. But now this branch of engineering has developed considerably and include many other activities like design, regulation, planning and administrative functions. In municipal organizations a full fledged traffic engineering unit can be entrusted to look after public safety. Such a traffic engineering unit may have several division such as :

(a) Field studies

(b) Accident analysis

(c) Traffic control devices

(d) Design and planning

(e) Special investigations

(f) Economic analysis and decision theory in engineering design, and

(g) Administration

Traffic engineering units should have a proper place in highway departments or Public Works Department of the States. The financing for traffic engineering activities is another problem. Obviously, the travelling public is more concerned about their safe and quick movements and hence a provision can be made to divert part of the income obtained from the road users in the form of taxes, parking charges, tolls etc. towards these activities.

5.7.2 Urban Transportation Planning Process

The transportation planning process is developed in a series of stages :

(i) Inventories

(ii) Trip generation

(iii) Trip distribution

(iv) Model split

(v) Traffic assignment

(vi) Plan preparation and evaluation

Inventories

Information related to land use, economic activity, population, travel characteristics and transportation facilities are collected through a series of surveys. For this purpose the metropolitan area under study is sub-divided into a number of smaller zones as shown in Fig. 5.7. The following rules are normally followed for zoning :

(i) Zones should be homogeneous in land use.

(ii) Zones should be of homogeneous traffic generating characteristics.

(iii) Zones should conform to enumeration districts, natural and physical barriers.

(iv) Zones should not be large enough to produce errors resulting from the assumption that all activities occur at zonal centroid,

(v) Zones should preferably have a geometrical shape for easy determination of centroids.

Detailed surveys are than organised to assess the existing activity levels and transportation facilities. Normally Home interviews surveys, Population data, Home hold trends. Socio-economic characteristics of the population, Land use and economic activities, Traffic volume census; Travel time studies and a Physical inventory of highway net work are carried out. Some of the surveys are explained in Article 5.2.3.

The information collected are analysed with respect to past trends and future expectations forming the basis for further travel demand analysis.

Trip Generation

This is the first stage of the travel demand forecasting process. Trip generation concerns with the estimation of number of trips produced in or attracted to a given zone. The *Trip* is defined as the "One-way movement having single purpose and mode of travel between a point of origin and a point of destination". Two popular methods of trip generation estimation are

Multiple Regression Analysis

Category Analysis

In Multiple Regression Analysis a functional relationship is expressed between the trips per zone and various socioeconomic activity levels in that zone. It is usually a linear model of the form :

$$y = b_0 + b_1 x_1 + \dots \dots \dots + b_k x_k \qquad (5.28)$$

where y = dependent variable-trips produced or attracted in a zone.

$x_1, x_2, \dots \dots x_k$ = independent variables that cause generation of trips.

$b_0, b_1, \dots \dots b_k$ = regression coefficients which are to be calibrated from the base year data obtained in inventory studies.

As an example in one of the Indian cities, the following relationship was established :

$$y = 1197.32 + 0.0957 x_2 \qquad (5.29)$$

where x_1 = number of workers in the zone

x_2 = number of vehicles in the zone

and y = number of trips produced for work purposes in a zone

On the assumption that this model remains stable over time, the future number of trips likely to be generated can be predicted by substituting the future estimates of the $x_1, x_2, \dots \dots x_k$, the independent variables in the equation.

In the category analysis, the household trip making is considered rather than zonal trip making. The procedure is to divide the households into a set of categories and to determine the base year trip rates to each category. In England 108 different categories are normally used based on 6 income classes, 3 car ownership levels, and 6 house hold structures. To predict the future trip generation, the expected number of households in such category at the design year are to be multiplied by the corresponding trip rate and summed up over the zone.

Trip Distribution

Trip distribution is the stage where the trips generated and attracted from each zone are distributed to any other zone. The most important method for this procedure is the *Gravity Model*. This model is based on the principle that the trips between any two zones i and j are directly proportional to the number of trips generated in the zone i, the number of trips attracted to zone j, and are inversely proportional to some function of distance or separation between the zones.

The model is as follows :

$$T_{ij} = \frac{G_i A_j F_{ij}}{\sum_{j=1}^{n} A_j F_{ij}} \qquad (5.30)$$

Here T_{ij} = number of trips from zone i to zone j

G_i = Trips generated in zone i

A_j = Trips attracted to zone j

F_{ij} = Empirically derived 'friction factor' calculated on area wise basis

n = number of zones in the urban area.

Existing data is used initially to calibrate the mode parameters F_{ij} through a computer program. Assuming these parameters to be same at a future date, the future trip interchanges are computed by substituting the future trip generated values in the model.

Model Split

The proportion of total trips between any two zones that can be shared between the private vehicles and the public transportation system is determined in this stage. The models so far developed have been designed to determine this proportion between car and bus modes. Several approaches are available to solve this problem. Typically the times and costs of travel by car mode and bus mode are assessed between two O-D points. With the help of diversion curves the number of bus trips likely to be made within the O-D pair is then determined.

Traffic Assignment

The next stage in the transportation planning is the assignment of various trips between any two O-D pairs on different highway routes. A typical method is known as "All-or-Nothing" procedure. In this the shortest route between the given O-D pair is identified. All the trips between the given O-D pair is identified. All the trips between these two zones are then assigned to this path. Similarly the trips between any two O-D pairs are assigned to respective shortest routes. A number of other complex methods are also available and give more realistic traffic assignments.

Plan Preparation and Evaluation

The steps described above will enable the various land use strategies and travel demands to be explored. In very general terms, a set of objectives are specified and various land use and transportation plans are developed. The benefits and losses likely to be caused by any alternative and the net returns with the estimated investment are then compared. An economic criterion is used to choose the best alternative by a critical analysis.

Example 5.17

The following information was obtained from a transportation survey of a town :

Traffic zone number	Population in the zone (thousands)	Total trips generated (in hundreds)
1	26	12
2	28	11
3	31	17
4	33	15
5	22	12
6	30	15
7	20	9
8	25	13

Develop a linear regression model for estimating the trips generated from a zone. If the population in a particular zone increases to 40,000, predict the expected trip generation from that zone.

Solution

(a) In this case there is one independent variable population and the problem is to develop a linear equation of the type.

$$y = b_0 + b_1 x_1$$

Here y = Total number of trips in hundreds per zone, being the dependent variable.

x_1 = Population of a zone in thousands, being the independent variable.

b_0 = Regression constant or intercept term.

b_1 = Regression coefficient

The equation is calibrated for b_0 and b_1 by the following formulae :

$$b_1 = \frac{n \sum xy - \sum x \sum y}{n \sum x^2 - (\sum x)^2} \qquad (5.31)$$

Also the correlation coefficient r, which tells about the goodness of fit is obtained by :

$$r = b_1 \left[\left(\frac{n \sum x^2 - (\sum x)^2}{n \sum y^2 - (\sum y)^2} \right) \right]^{1/2} \qquad (5.32)$$

Here n = total number of observations

Σ = Summation of the variable over all the observations

It is convenient to prepare a tabular form for computations :

Zone	x	y	xy	x^2	Y^2
1	26	12	312	676	144
2	28	11	308	784	121
3	31	17	527	961	289
4	33	15	495	1089	225
5	22	12	264	484	144
6	30	15	450	900	225
7	20	9	180	400	81
8	25	13	325	625	169
n = 8	$\Sigma x = 215$	$\Sigma y = 104$	$\Sigma xy = 2861$	$\Sigma x^2 = 5919$	$\Sigma y^2 = 1398$

Substituting these values in the formulae given earlier

$$b_1 = \frac{8 \times 2861 - 215 \times 104}{8 \times 5919 - (215)^2} = 0.469$$

$$b_0 = (104 - 0.469 \times 215)/8 = 0.396$$

Therefore, the trip generation model is

$$y = 0.396 + 0.469\,x$$

The correlation coefficient for this model is

$$r = 0.469 \left[\left(\frac{8 \times 5919 - (215)^2}{8 \times 1398 - (104)^2} \right) \right]^{1/2} = 0.82$$

The linear regression model is given by :

$$y = 0.396 + 0.469\,x,\ r = 0.82$$

(b) The future population of a zone = 40,000

x = 40 for use in model

The total trips generated, y = $0.396 + 0.469 \times 40 = 19.16$ in hundreds = 1916

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PROBLEMS

1. What are the objects and scope of traffic engineering ? Explain briefly.
2. What is the significance of road user characteristics in traffic engineering ? Discuss briefly the various factors which affect the road user characteristics and their effects in traffic performance.
3. What are different vehicular characteristics which affect the road design ? Briefly explain.
4. Indicate the maximum dimensions and weight of vehicles allowed in India, as specified by I.R.C. Discuss the effect of wider vehicles on the road.
5. Discuss the various traffic studies and their importance.
6. Explain the term traffic volume. What are the objects of carrying out traffic volume studies ?
7. Enumerate the different methods of carrying out traffic volume studies. Indicate the principle of each.
8. Indicate how the traffic volume data are presented and the results used in traffic engineering.
9. Explain spot speed, running speed, space-mean speed time-mean speed and average speed. How are spot speed studies carried out ?
10. What are the various objects and applications of spot-speed studies ?
11. Explain how the speed and delay studies are carried out. What are the various uses of speed and delay studies ?
12. Explain origin and destination study. What are the various uses of O & D studies.
13. Explain the traffic manoeuvres and their applications.
14. Explain traffic capacity, basic capacity, possible capacity and practical capacity.
15. Discuss briefly the various factors affecting the practical capacity of road.
16. Estimate the basic capacity of traffic lane at a speed of 60 kmph. Assume that all the vehicles are of average length 6 m.
17. Write short notes on :
    (a) Thirteenth highest hourly traffic volume
    (b) Desire lines

(c) Traffic signal system

(d) Before and after studies

(e) PCU

(f) Level of service

18. (a) Explain the relationship between speed, travel time, volume, density and capacity.
(b) What are the factors on which PCU values depend ?
(c) Explain the level of service concept while deciding the design capacity of a road.
19. Explain briefly the various aspects investigated during parking studies. What are the uses of these studies ?
20. What are the different causes of traffic accidents ? Discuss briefly.
21. What are the applications of location file, spot, maps, collision diagrams and condition diagrams ?
22. Explain various measures that may be taken to prevent accidents.
23. What are the various types of traffic accidents ? Discuss the method of analyzing the speed of vehicle involved in the accident.
24. A vehicle skids through a distance equal to 40 m before colliding with another parked vehicles, the weight of which is 75 per cent of the former. After collision if both the vehicles skid through 14 m before stopping compute the initial speed of moving vehicle. Assume friction coefficient of 0.62.
25. Two vehicles A and B of equal weight, approaching from cross roads (at right angles) collide with each other. They skid through distances 30 m and 20 m before collision and 20 m and 35 m respectively after collision. If the directions of skidding vehicles A and B after collision are 45° and 130° with original path respectively, calculate the original speeds of the two vehicles before the application of brakes. Assume f = 0.55.
26. With neat sketches show various types of traffic signs., classifying them in proper groups.
27. What are the advantages and disadvantages of traffic signals ?
28. Explain the various types of traffic signals and their functions. How are the signal timings decided ?
29. Design the timings of an isolated signal to be installed at a right angled intersection when roads P and Q cross. The data available are :

	Road P	Road Q
Width, metre	14.0	10.5
Peak hour traffic volume, vehicles per hour per lane	200	120
Approach speed, kmph	50	35

30. What are the various types of traffic markings commonly used ? What are the uses of each ?
31. What are the various types of traffic islands used. Explain the uses of each.

32. Enumerate the various types of intersections and the basic principles involved.
33. With neat sketches show few typical patterns of unchannelized and channelized intersections. What are the advantages and limitations of unchannelized and channelized intersections ?
34. What is a traffic rotary ? What are its advantages and limitations, in particular reference to traffic conditions in India ?
35. Explain clearly how the actual crossing manoeuvre of traffic is avoided in a traffic rotary though the traffic may have to otherwise go in cross directions of radiating roads.
36. Explain briefly the various design factors that are to be considered in rotary intersection design.
37. Explain grade separated intersections, the advantages and limitations.
38. What are the relative advantages and disadvantages of over-pass and under-pass ?
39. Draw a neat sketch of a full cloverleaf and show the movement of traffic.
40. What are the various types of parking facilities designed for traffic needs ? Compare kerb parking with off-street parking.
41. Compare (a) angle parking with parallel parking (b) parking lots and garages (c) ramp type and elevator type parking garages.
42. What are the objects of highway lighting ? Explain silhoutte and reverse silhoutte.
43. Explain the various design factors in road lighting.
44. Illustrate with sketches recommended types of lights and luminaire distribution.
45. Write a note on lighting layouts for highway and intersections.
46. Calculate the spacing between the lighting units to produce a lux equal to 7.0 from the following data :

Width of road = 14 m

Mounting height = 8 m

Lamp size = 7000 lumen

Luminaire type II.

47. Briefly explain the objectives of traffic planning and administration.
48. The driver of a vehicle approaching a signalized intersection at a speed of 40 kmph applied brakes on seeing the signal changing from green to amber and the vehicle was brought to top on the prescribed stop line during the amber time of 4 seconds. If the reaction time of the driver is assumed as 1.0 second, compute the average friction coefficient developed.
49. In a road test for measuring skid resistance using skid resistance equipment, the timer indicating 4.25 seconds of brake application and the braking distance indicated by the colour spray was measured as 32.3 metre before the vehicle was brought to stop. What is the average skid resistance of the surface ?
50. A driver travelling at 50 kmph behind another car decides to pass it and accelerate. If the rate of acceleration 'a' is given by the relation $\frac{dv}{dt} = (1.2 - 0.015\ v)$ where v is the speed m/sec. and t is time in seconds, find

(a) the rate of acceleration after 8 seconds

(b) time taken to attain a speed of 100 kmph.

51. The driver of a vehicle travelling at 80 kmph up a grade required 9 m less to stop after he applies the brakes than the driver travelling at the same initial speed down the same grade. If the coefficient of friction between tyre and pavement is 0.5, what is the percent grade and what is the braking distance down the grade ?

52. The following data were obtained from the spot speed studies carried out at a city road during a certain period of time. Suggest (a) speed limit for regulation (b) Speed to check geometric design elements (c) Lower speed group causing congestion (d) Median speed (e) Dispersion.

Speed group kmph	No. of vehicles	Speed group kmph	No. of vehicles
< 5	45	30 – 35	430
5 – 10	230	35 – 40	290
10 – 15	375	40 – 50	110
15 – 20	500	50 – 60	25
20 – 25	680	60 – 70	8
25 – 30	525	> 70	2

53. The relationship between the hourly traffic volume as percentage of AADT and number of hours in a year when the traffic volume exceeds is as given below for a road.

No. of hrs. exceeding	1	5	10	20	30	60	90	150	300
% AADT	24	20	18	16	156	15	14	13	12

If the AADT projected for the design period is 8,500 estimate the design hourly volume justifying your results.

54. Show the conflict points at the intersection of the following types :

(a) Cross-roads, both two-way

(b) Cross-roads, one-way

(c) T-intersection both two-way

(d) Y-intersection, one one-way

55. On cross roads A and B the 15 minutes traffic volume during the design hour were 700 and 400 vehicles. The approach speeds were 50 and 30 kmph for roads A and B. The width of road A is 14 m and that of road B is 10 m. Design the signal timings.

(a) By trial method

(b) By simple method with pedestrian signals.

56. A van of weight 3 T hits a parked car of weight 0.8 T and both the vehicles skid together through a distance of 4.2 m before coming to stop. Calculate the initial speed of the van if, (i) it does not apply brakes before collision (ii) it applies brakes and skids through a distance of 2.8 metre before collision.

57. Two trucks A and B of gross weight 7 T and 12 T approaching from right angles applied brakes and skid through distances 3.2 m and 1.9 m respectively before collision. After collision truck A was thrown back making an angle of 50° with its original direction and skid through a distance of 2.8 m. Truck B skid along a distance of 3.8 m deviating at an angle of 60° from the original path. Estimate the initial speeds of approach of the two trucks.

58. The following Table provides the land use and trip data. Develop a model for trip attraction : Compute the correlation coefficient.

Zone No.	Total Employment	Trips Attracted
1	9482	9428
2	2010	2192
3	574	330
4	127	153
5	3836	3948
6	953	1188
7	223	240
8	36	55
9	2223	2064
10	272	280

59. Develop a model for the following data :

Education Trips	10	12	17	16	18	20
School Population	20	24	28	32	36	39

Chapter 6

Highway Materials

6.1 SUBGRADE SOIL

6.1.1 Significance of Subgrade Soil

Subgrade soil is an integral part of the road pavement structure as it provides the support to the pavement from beneath. The subgrade soil and its properties are important in the design of pavement structure. The main function of the subgrade is to give adequate support to the pavement and for this the subgrade should possess sufficient stability under adverse climate and loading conditions.

The formation of waves, corrugations, rutting and shoving in black top pavements and the phenomena of pumping, blowing and consequent cracking of cement concrete pavements are generally attributed due to the poor subgrade conditions.

When soil is used in embankment construction, in addition to stability incompressibility is also important as differential settlement may cause failures. Compacted soil and stabilized soil are often used in sub-base or base course of highway pavements. The soil is therefore considered as one of the principal highway materials.

6.1.2 Characteristics of Soil

Soil consists mainly of mineral matter formed by the disintegration of rocks, by the action of water, frost, temperature, pressure or by plant or animal life. Based on the individual grain size of soil particles, soils have been classified as gravel, sand, silt and clay. The characteristics of soil grains depend on the size, shape, surface texture, chemical composition and electrical surface charges. Moisture and dry density influence the engineering behaviour of a soil mass.

6.1.3 Desirable Properties

The desirable properties of soil as a highway material are

(i) Stability

(ii) Incompressibility

(iii) Permanency of strength

(iv) Minimum changes in volume and stability under adverse conditions of weather and ground water

(v) Good drainage, and

(vi) Ease of compaction.

The soil should possess adequate stability or resistance to permanent deformation under loads, and should possess resistance to weathering, thus retaining the desired subgrade support. Minimum variation in volume will ensure minimum variation in differential expansion and differential strength values. Good drainage is essential to avoid excessive moisture retention and to reduce the potential frost action. Ease of compaction ensures higher dry density and strength under particular type and amount of compaction.

6.1.4 Index Properties of Soil

The wide range of soil types available as highway construction materials have made it obligatory on the part of the highway engineer to identify and classify the different soils. The soil properties on which their identification and classification are based are known as *index properties*. The index properties which are usually used are *grain size distribution, liquid limit and plasticity index*. Further the properties which are some times used are *shrinkage limit, field moisture equivalent centrifuge moisture equivalent and compacted dry density*.

Grain Size Analysis

The grain size distribution is found by mechanical analysis. The components of soils which are coarse grained may be analysed by *sieve analysis* and the soil fines by *sedimentation analysis*. The grain size analysis or the mechanical analysis is hence carried out to determine the percentage of individual grain size present in a soil sample.

The sieve analysis is a simple test consisting of sieving a measured quantity of the material through successively smaller sieves; the weight retained on each sieve is expressed as a percentage of the total weight of sample. The sedimentation principle, that the larger grains in a suspension settle faster, is used for finding the grain size distribution of fine soil fraction passing 75 micron sieve. Two methods of test viz. : Hydrometer method and Pipette method are used based on sedimentation principle. The details of the grain size analysis tests as well as all tests on highway materials have been given by the authors in their book, *Highway Material Testing*.

Consistency Limits and Indices

The physical properties of fine grained soils, especially of clays differ very much at different water contents. A clay may be almost in a liquid state, or it may show plastic behaviour or may be stiff depending on the moisture content. Plasticity is a property of outstanding importance for clayey soils, which may be explained as ability to undergo changes of shape without rupture. *Atterberg* in 1911 proposed a series of tests, mostly empirical, for the determination of the consistency and plastic properties of fine soils. These are known as *Atterberg limits* and *indices*.

Liquid limit may be defined as the minimum water content at which the soil will flow under the application of very small shearing force. The liquid limit is usually determined in the laboratory using a mechanical device.

Plastic limit may be defined in general terms, as the minimum moisture content at which the soil remains in a plastic state. This lower limit is arbitrarily defined and determined in the laboratory by a prescribed test procedure.

Plasticity index (P.I.) is defined as the numerical difference between the liquid and plastic limits. Plasticity index thus indicates the range of moisture content over which the soil is in plastic condition.

Shrinkage limit is the maximum moisture content at which further reduction in water content does not cause reduction in volume. It is the lowest water content that can occur in clayey soil sample which is completely saturated.

Consistency limits and the plasticity index vary for different soil types and therefore these properties are generally used in the identification and classification of soils. Generally soils having high values of liquid limit and plasticity index are poor as engineering materials. Both liquid limit and plastic limit depend on the type and amount of clay in soils. The plasticity index generally depends only on the amount of clay present; giving an indication of clay content in soil. In soil having same values of liquid limit, but with different values of plasticity index; it is generally found that rate of volume change and dry strength increases and permeability decreases with increase in plasticity index. In soils having same values of plasticity index but different values of liquid limit, it is seen that compressibility and permeability increase and dry strength decreases with increase in liquid limit. Thus the values of liquid limit and plasticity index help in classifying the cohesive soils.

In addition to the above tests certain other properties have also been some time used in identifying and classifying soils. These include shrinkage limit, field moisture equivalent, centrifuge moisture equivalent and compaction characteristics of the soils.

Field moisture equivalent of a soil is the moisture content at which the demands for absorbed water are fully satisfied. The *centrifuge moisture equivalent* of a soil is the moisture content retained against a force of 1000 times gravity for one hour. These tests are seldom carried out now-a-days. In most of the soil classifications systems that are commonly in use, the classifications are based on the grain size distribution (by sieve analysis), liquid limit and plasticity index of the soils.

6.1.5 Soil Classification Based on Grain Size

There are several classification systems for soil grains based on grain size of soil, according to which soils have been classified as gravel, sand, silt and clay. The exact limits of grain size for each of these components are not same in all these classifications. The most widely accepted grain size classification system is the *M.I.T. classification system*. The Indian Standards Institution (I.S.I.) has also adopted the same limits of M.I.T. system for the *Indian Standard Classification System* of soil grains. The limits of the grain size for each component as per this system are shown below :

Gravel	Sand			Silt			Clay		
	Coarse	Medium	Fine	Coarse	Medium	Fine	Coarse	Medium	Fine or Colloidal
	2.0*	0.6	0.2	0.06	0.02	0.006	0.002	0.0006	0.0002

*Values are in mm

Textural Classification

The textural classification system is based on grain size distribution of the soil and is helpful in classifying a soil which contains different soil components such as sand silt and clay. A typical textural classification chart for subgrade soils (sand and smaller grain sizes (suggested by *U. S. Bureau of Public Roads*) is given in Fig. 6.1.

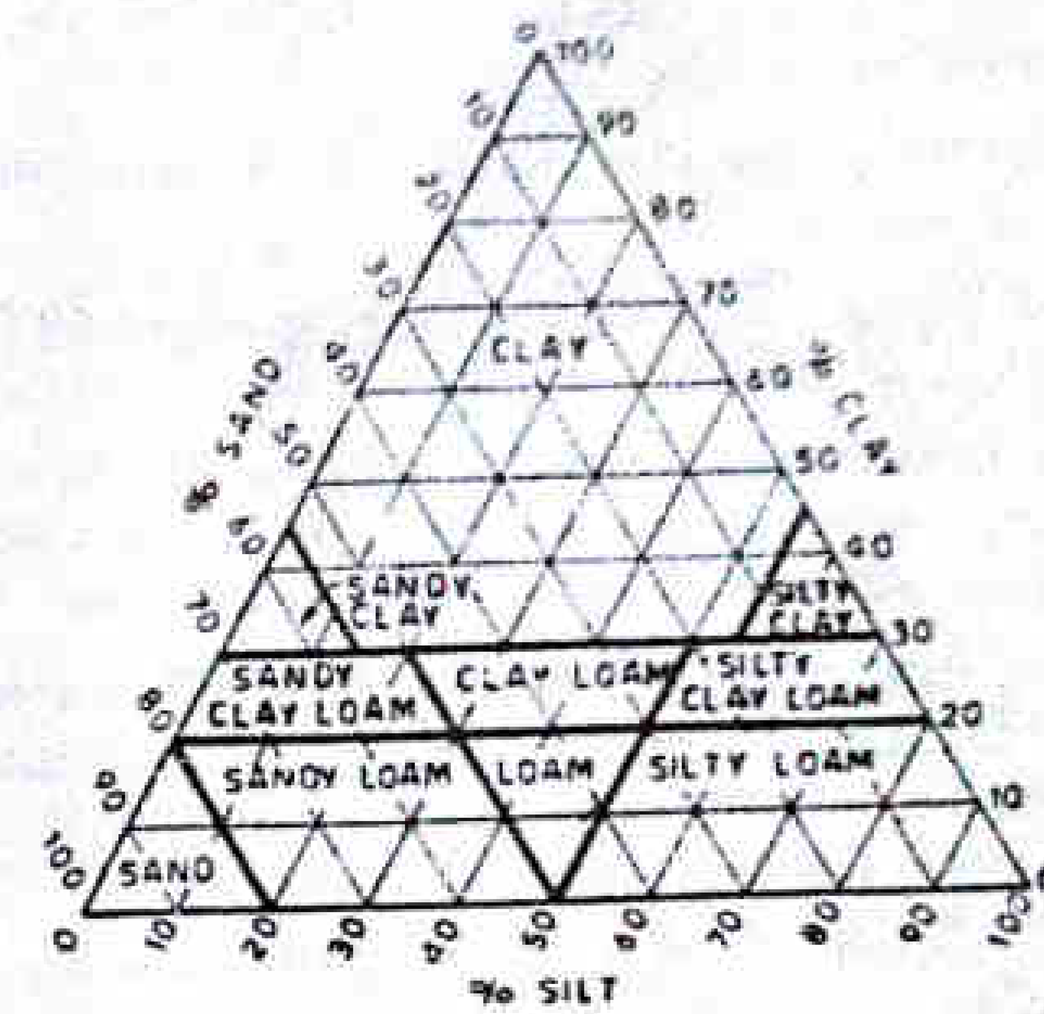


Fig. 6.1 Textural Classification Chart

6.1.6 Soil Classification Systems

The various soil classification systems in use in the field of highway engineering are :

(i) Burmister descriptive classification

(ii) Casagrande soil classification

(iii) Unified soil classification of Revised Casagrande soil classification and I. S. soil classification systems.

(iv) U. S. Public Roads Administration (PRA) classification

(v) Highway Research Board (HRB) or American Association of State Highway Officials (AASHO) classification or Revised PRA classification.

(vi) Federal Aviation Agency (FAA) classification.

(vii) Civil Aeronautic Administration (CAA) classification.

(viii) Compaction classification.

Of these systems, the Unified soil classification system has been very widely accepted in general for the classification of soils for civil engineering purposes and the H.R.B. classification or the revised PRA system is adopted for the classification of subgrade soils in Highway Engineering. Hence these two classification systems are given here :

Unified soil classification system

The Casagrande classification system which was developed in 1942 to classify soils, was later revised, modified and adopted by both the U. S. Crops of Engineers and the U. S. Bureau of Reclamation and was re-named as *Unified Soil Classification System*.

The soils are divided into two board groups, coarse grained and fine grained, based on grain size. The coarse grained soils include gravels (groups symbol G) and sand (group symbol S). Each of these component have been subdivided as well graded (symbol W), well graded with clay binder (symbol C), poorly graded (symbol P) and material containing considerable proportion of silt (symbol M). Thus a well graded gravel is GW, well graded sand SW, poorly graded gravel GP and so on.

The fine grained soil with more than half passing 200-mesh I. S. sieve (0.074 mm size) have been divide into two groups :

(i) Soils with liquid limit less than 50 percent or soils with low to medium compressibility (group symbol L).

(ii) Soils with liquid limit greater than 50 percent or soils with high compressibility (group symbol H).

Symbols M, C and O have been assigned to inorganic silts including very fine sand, inorganic clay and organic soils (silt and clay) respectively. Thus inorganic silt with low plasticity is ML, inorganic silt with high plasticity is MH and so on.

Unified soil classification groups, symbols and limits of test properties are given in Table 6.1. This soil classification system makes use of the results of sieve analysis, liquid limit and plastic limit tests. The classification group of the fine grained soil is found by making use of plasticity chart given in Fig. 6.2.

Table 6.1 Unified Soil Classification

Major Division			Symbol	Brief description of soil types	Laboratory test results
Coarse grained soils, more than 50% material larger than No. 200 sieve size (0.074 mm)	Gravel, "G" more than half of coarse fraction larger than 4.76 mm sieve	Clean gravel with little or no fines* (less than 5%)	GW	Well graded gravels and gravels sand mixtures	Uniformity coefficient- $Cu = D_{60}/D_{10} > 6$ Gradation coefficient, $Cg = D^2_{30}/D_{60} D_{10} = 1$ to 3
			GP	Poorly graded gravels and gravel -sand mixtures	Not meeting the Cu and Cg requirement of GW
		Gravel with appreciable proportion of fines* (more than 12%)	GM	Silty gravel and gravel sand mixtures	
			GC	Clayey gravels and gravel-sand-silt mixtures	
	Sands, "S" more than half of coarse fraction smaller than 4.76 mm	Clean sand with little or no fines* (Less than 5%)	SW	Well graded sand and gravelly sands	Cu < 4 Cg = 1 to 3
			SP	Poorly graded sands and gravelly sand	Not meeting the Cu and Cg requirements of SW
		Sands with appreciable proportion of fines* (more than 12%)	SM	Silty sand and sand-silt mixtures	
			SC	Clayey sand and sand clay mixtures	
Fine grained soils, more than 50% materials smaller than No. 200 sieve size (0.074 mm)	Silts and clays with liquid limit less than 50 – "L"		ML	Inorganic silts, very fine rock floor, clayey silt or fine sand	Classification by plasticity chart (Fig. 6.2)
			CL	Inorganic clays, gravelly sandy or silty	
			OL	Organic silt and silty clays	
	Silts and clays with liquid limit greater than 50 – "H"		MH	Inorganic silt elastic and Micaceous silts	
			CH	Inorganic fat clays	*Fines are those materials smaller than No. 200 sieve size or 0.074mm size
			OH	Organic silt and clays	
	Highly organic soils		Pt	Peat and other highly organic soils	

I.S. soil classification

The Indian Standard Institute (ISI) has also adopted a soil classification system based on the Unified Soil Classification System. There is only slight variation in some of the subgroups and their symbols. The particulars of IS soil classification is given in Table 6.2.

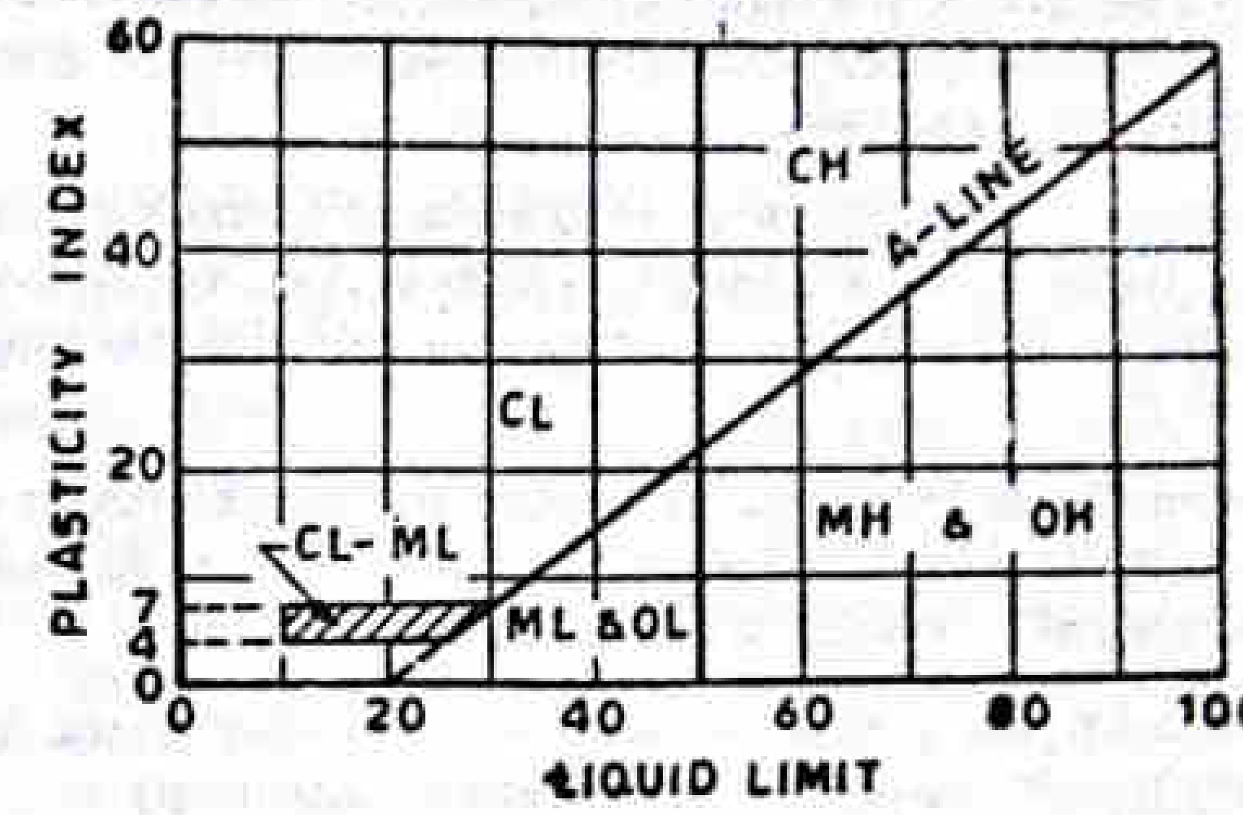


Fig. 6.2 Plasticity Chart

Table 6.2 I. S. Soil Classification

Division and	Sub-Division	Soil Group Description	Symbol
Coarse - grained soil (more than half of the total material larger than IS sieve 8)	Gravelly soils more than half grains larger than IS sieve 480	Well-graded gravel or gravel sand mixtures, with clay binder	GB
		Well-graded gravels or gravel sand mixtures with little or no fines.	GW
		Clayey gravels poorly graded gravel-sand-clay mixtures	GC
		Silty-gravel or poorly graded gravel-sand-silt mixtures	GM
		Poorly graded gravels or gravel sand mixtures with little or no fines	GP
	Sandy soils, more than half of the coarse grains smaller than IS sieve 480	Well-graded sand or gravelly sands, with clay binder	SB
		Well-graded sand or gravelly sands with little or no fines	SW
		Clayey sand poorly graded sand-clay mixtures	SC
		Silty sands or poorly graded sand-silt mixtures	SM
		Poorly graded sands or gravelly sands with little or no fines	SP
Fine grained inorganic soils (more than half of the total material smaller than IS sieve 8)	Inorganic silts and clays with low or Medium compressibility	Silt and very fine sand; rock flour; silty to clayey fine sands with low plasticity	ML
		Gravelly clays, sandy clays, silty clays, lean clays of low plasticity	CL
		Clay of medium plasticity	CI
	Inorganic silts and clays with high compressibility	Very compressible micaceous or diatomaceous fine silty soils, silts	MH
		Clays of high plasticity	CH
Silt and clay with high organic content	Silts, organic	Silty and Silt-clays of low plasticity	OL
	Clays, organic	Clays of medium to high plasticity very compressible	OH
Peat	Peat	Peat and other highly organic swamp soils	Pt

Highway Research Board (HRB) classification of soils

This is also called American Association of State Highway Officials (AASHO) classification of Revised Public Roads Administration (PRA) soil classification system. The original soil classification was developed by Bureau of Public Roads in 1928. After about 15 years of experience, certain revisions and modifications were made and the HRB classification system was developed. In fact by these modifications it is now possible to have subgroups for properly classifying different soil types and the number of classification tests were decreased from six to three, thus enabling classification of soils by three simple laboratory tests namely, sieve analysis liquid limit and plastic limit.

Soils are divided into seven groups A-1 to A-7. A-1, A-2 and A-3 soils are granular soils, percentage fines passing 0.074 mm sieve being less than 35. A-4, A-5, A-6 and A-7, soils are fine grained or silt-clay soils, passing 0.074 mm sieve being greater than 35 percent.

A-1 soils are well graded mixture of stone fragments, gravel coarse sand, fine sand and non-plastic or slightly plastic soil binder. The soils of this group are subdivided into two subgroups, A-1-a, consisting predominantly of stone fragments or gravel and A-1-b consisting predominantly of coarse sand.

A-2 group of soils include a wide range of granular soils ranging from A-1 to A-3 groups, consisting of granular soils and upto 35% fines of A-4, A-5, A-6 or A-7 groups. Based on the fines content, the soils of A-2 groups are subdivided into subgroups A-2-4, A-2-5, A-2-6 and A-2-7.

A-3 soils consist mainly, uniformly graded medium or fine sand similar to beach sand or desert blown sand. Stream-deposited mixtures of poorly graded fine sand with some coarse sand and gravel are also included in this group.

A-4 soils are generally silty soils, non-plastic or moderately plastic in nature with liquid limit and plasticity index values less than 40 and 10 respectively.

A-5 soils are also silty soils with plasticity index less than 10%, but with liquid limit values exceeding 40%. These include highly elastic or compressible, soils, usually of diatomaceous of micaceous character.

A-6 group of soils are plastic clays, having high values of plasticity index exceeding 10% and low values of liquid limit below 40%; they have high volume change properties with variation in moisture content.

A-7 soils are also clayey soils as A-6 soils, but with high values of both liquid limit and plasticity index, (LL greater than 40% and PI greater than 10%). These soils have low permeability and high volume change properties with changes in moisture content.

Groups Index of Soil

Fine grained soils of each classification group exhibit a wide range of properties as subgrade material.

In order to classify the fine grained soils within one group and for judging their suitability as subgrade material, an indexing system has been introduced in HRB classification which is termed as *Group Index*. Soils are thus assigned arbitrary numerical numbers known as group index (GI). Group index is function of percentage material passing 200 mesh sieve (0.074 mm), liquid limit and plasticity index of soil and is given by the equation :

$$GI = 0.2\,a + 0.005\,a\,c + 0.01\,b\,d \qquad (6.1)$$

Here, a = that portion of material passing 0.074 mm sieve, greater than 35 and not exceeding 75 percent (expressed as a whole number from 0 to 40)

b = that portion of material passing 0.074 mm sieve greater than 15 and not exceeding 35 percent (expressed as a whole number from 0 to 40)

c = that value of liquid limit in excess of 40 and less than 60 (expressed as a whole number from 0 to 20)

d = that value of plasticity index exceeding 10 and not more than 30 (expressed as a whole number from 0 to 20)

According to this formula, the minimum possible value of group index is zero and the maximum possible value is 20, when the values of soil fraction passing 0.074 mm sieve, liquid limit and plasticity index are respectively higher than 75, 60 and 30 percent. Higher the value of group index, poorer is the soil as subgrade material.

The sub-groups A-2-6 and A-2-7 soils of A-2 group have GI values 0 to 4, A-4 group of soil have GI values up to 8; A-5 soil up to 12, A-6 soil up to 16 and A-7 soil up to 20. The group index value is indicated as suffix to the soil group within brackets, such as A-6 (4) or A-6 (16). In this example an A-6 soil with group index value 4 is considered as superior subgrade material than the A-6 soil with group index value 16.

The soil groups showing the classification limits of various properties is given in Table 6.3. In order to classify a soil, the values of test results are attempted to be fitted in from left column towards right side of Table 6.3 and the correct group is found by the process of elimination. The first group from the left to which the test data fits in gives the classification group. Figure 6.3 gives the chart for finding the group index value form the values of percent passing 0.074 mm sieve, liquid limit and plasticity index, instead of using the Eq. 6.1 for group index. Here the group index value is the sum of the values obtained on vertical axes from both the charts based on the values of percent passing 0.074 mm sieve, LL and PL Fig. 6.4 gives the chart for classifying fine grained soil from the liquid limit and plasticity index values. GI values have been made use of in the design of flexible pavement thickness in one of the empirical design methods as given in Art. 7.3.2.

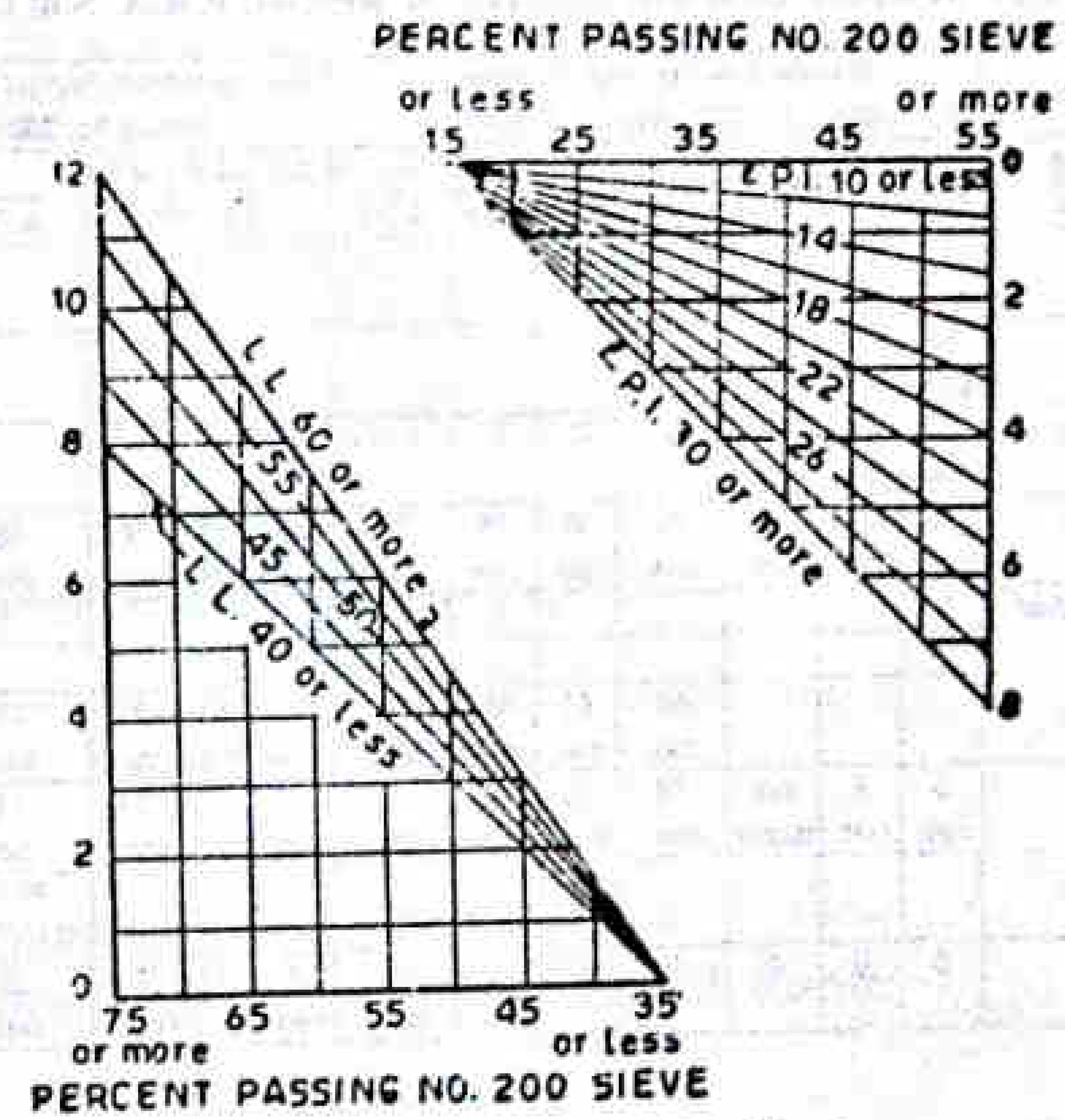


Fig. 6.3 Group Index Chart

Characteristics of Soil Classification Groups

The soil classification systems have gained importance in subsequent years, as the soils belonging to the different classification groups are qualified based on their stability or bearing capacity, drainage characteristics, potential frost action and volume change properties. The suitability of soils as subgrade material could also be judged by knowing the soil classification group. In Table 6.4, some of these properties of the unified soil classification groups are given. Table 6.5 gives some of the properties of the HRB soil classification groups. Thus soil classification is useful to predict some of the soil characteristics from simple physical tests.

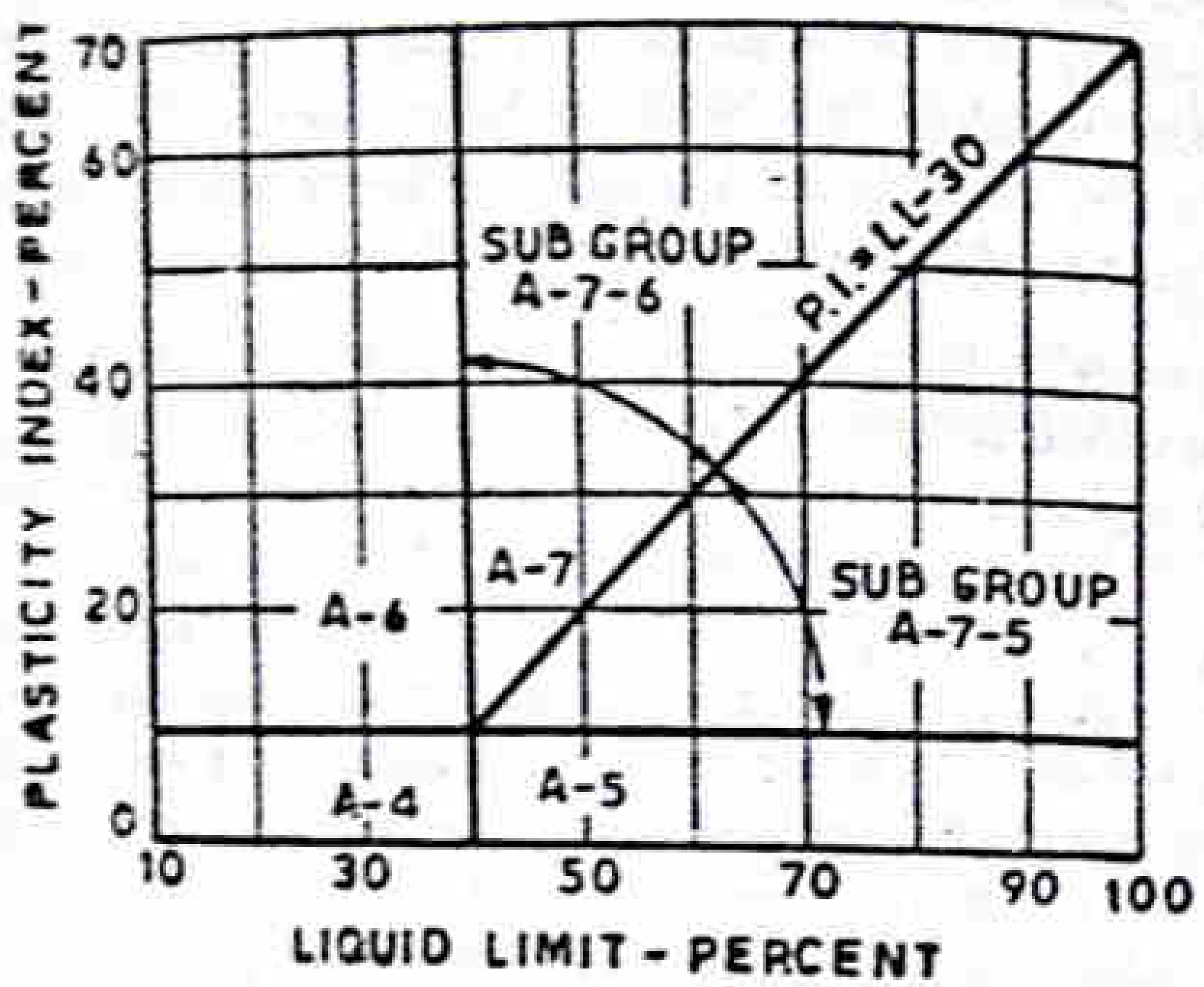


Fig. 6.4 Chart for Classifying Fine Grained Soil (H.R.B. system)

Table 6.3 Highway Research Board or AASHO or Revised P.R.A. Soil Classification

General Classification	Granular soils, less than 35 percent passing No. 200 sieve size (0.074 mm)							Fine grained (silt-clay) soils, more than 35% passing No. 200 sieve size				
Classification Group	A-1		A-3	A-2				A-4	A-5	A-6	A-7	
Sub classification	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5	A-7-6
Sieve analysis, % passing												
No. 10 sieve (2 mm size)	50 max.											
No. 40 sieve (0.42 mm size)	30 max.	50 max.	51 min.									
No. 200 sieve (0.074 mm size)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction Passing no. 40 sieve												
Liquid limit	-	-	-	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	41 min.
Plasticity index	6 max.	6 max.	non plastic	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.	11 min.
											PI < (LL – 30)	PI > (LL – 30)
Group Index	0	0	0	0	0	4 max.	4 max.	8 max.	12 max.	16 max.	20 max.	20 max.

Example 6.1

The results of sieve analysis of a soil are given below :

Sieve size, mm	Percent passing
4.76	60
2.00	30
0.60	10
0.40	5
0.20	0

(a) Classify the soil by (i) Unified and (ii) HRB soil classification systems.

(b) Discuss the suitability of the soil as a subgrade material.

Solution

(a) *Soil classification*

(i) By Unified System

Refer Table 6.1. As more than half (60%) is finer than 4.76 mm sieve and there are no fines the soil is sand or SW or SP groups.

$$\text{Uniformity coefficient } C_u = \frac{D_{60}}{D_{10}} = \frac{4.76}{0.6} = 7.9$$

$$\text{Gradation coefficient } C_g = \frac{(D_{30})^2}{D_{60}\,D_{10}} = \frac{2.0^2}{4.76 \times 0.6} = 1.7$$

As C_u is greater than 4 and C_g is between 1 and 3, the classification group of the sand is SW.

(ii) By HRB System

From Table 6.3, it is found that the classification group is A-1-a.

(b) Suitability as Subgrade Material

Refer Tables 6.4 and 6.5

Drainage	excellent
Volume change	almost none
Potential frost action	none to very slight
Stability	high
Values as subgrade material	very good

Example 6.2

The properties of subgrade soil are given below :

Passing 0.074 mm sieve = 55%

Liquid limit = 50%

Plastic limit = 41%

(a) Classify the soil by revised PRA/HRB system

(b) Discuss the suitability of the soil as a subgrade material.

Solution

(a) Soil Classification

From Table 6.3, as percentage passing 0.074 mm sieve is more than 35, the soil is fine grained.

$$LL = 50\%$$

$$PI = LL - PL = 50 - 41 = 9$$

Using the chart (Fig. 6.4) the classification group is A-5.

$$\text{Group Index, GI} = 0.2\,a + 0.005\,ac + 0.01\,bd$$

Table 6.4 Some Characteristics of unified Soil Classification Groups

Soil Type	Soil Group	Value as foundation or subgrade material	Unit dry weight (I.S. light compaction) g/m³	CBR %	Subgrade modulus kg/cm³	Drainage characteristics	Volume change characteristics	Potential frost action
Coarse grained soils Gravelly soils (G)	GW	Excellent	2.00 – 2.24	60 – 90	> 8.33	Excellent	Almost none	None to very slight
	GP	Excellent to good	1.76 – 2.08	25 – 60	> 8.33	Excellent	None	None
	GM	Excellent to good	2.08 – 2.22	20 – 80	> 8.33	Fair to poor	Very slight	Slight to medium
	GC	Good	1.92 – 2.24	20 – 80	5.55 – 8.33	Poor	Slight	Slight to medium
Coarse grained soils-Sand soils (S)	SW	Good	1.76 – 2.08	20 – 60	5.55 – 8.33	Excellent	Almost none	None to very slight
	SP	Good to fair	1.59 – 1.92	10 – 30	5.55 – 8.33	Excellent	Almost none	None to very slight
	SM	Good	1.92 – 2.16	10 – 40	5.55 – 8.33	Fair to poor	Very slight	Slight to high
	SC	Fair to Good	1.68 – 2.08	15 – 50	5.55 – 8.33	Poor	Slight to medium	Slight to high
Fine grained soils with low Compressibility	ML	Fair to Poor	1.60 – 2.00	5 – 20	2.78 – 5.55	Fair to poor	Slight to medium	High to very high
	CL	Fair to Poor	1.60 – 2.00	5 – 15	2.78 – 5.55	Very poor	Medium	Medium to high
	OL	Poor	1.44 – 1.60	3 – 8	2.78 – 5.55	Poor	Medium to high	Medium to high
Fine grained soils with high compressibility	MH	Poor	1.28 – 1.60	3 – 8	2.78 – 5.55	Poor	High	Medium to high
	CH	Poor to very poor	1.44 – 1.76	3 – 5	1.39 – 2.78	Very poor	Very High	Medium
	OH	Poor to very poor	1.28 – 1.68	2 – 4	1.39 – 2.78	Very poor	Very High	Medium

$$a = 55 - 35 = 20$$

$$b = 55 - 15 = 40$$

$$c = 50 - 40 = 10$$

$$d = 9 - 10 = 0 \text{ (min)}$$

$$\therefore GI = 0.2 \times 20 + 0.005 \times 20 \times 10 + 0.01 \times 40 \times 0$$

$$= 4.0 + 1.0 + 0 = 5.0$$

Alternatively GI may also be found using charts (Figure 6.3) and is found to be GI = 5.0 + 0 = 5.0.

Soil classification is A – 5 (5).

(b) Suitability as subgrade material

From Table 6.5,

Drainage	poor
Volume change	medium to high
Potential frost action	high to very high
Stability	poor
Values as subgrade material	poor

Fig. 6.5 Some Characteristics of H.R.B. Soil Classification Groups and Approximate Equivalent Group of Soil Classification System

Soil Group	Sub Group	General Stability Property and rating as subgrade material	Max. dry density (I. S. Light comp c/um³)	CBR %	Subgrade modulus kg/cm³	Drainage characteristics	Volume change characteristics	Potential frost action	Approximate equivalent unified classification
A-1	A-1-a	High stability very good to excellent subgrade	2.03 (min)	60 – 90	> 8.33	Excellent	Almost none	None to slight	GW, GP, GM
	A-1-b			20 – 70		Good			SW, SM
A-3		Stable when confined very good to fair subgrade material	1.29 – 2.03	10 – 80	> 5.55	Excellent	None	None	SP
A-2	A-2-4	Stable when dry; may ravel. Very good to good subgrade material	1.92 – 2.08	8 – 70	> 5.0	Good to Fair	Very slight	Slight to high	GM, SM
	A-2-5								
	A-2-6	Good stability. Very good to fair subgrade material				Fair to poor	Slight to medium	Very slight to medium	GC, SC
	A-2-7								
A-4		Satisfactory stability when dry. Loss of stability when wet or by frost action. Good to poor subgrade material	1.76 – 1.29	4 – 20	2.78 – 5.0	Fair to poor	Slight to medium	Very high	ML, OL
A-5		Difficult to compact doubtful stability, poor to very poor subgrade material	1.28 – 1.60	2 – 7	1.39 – 3.48	Poor	Medium to high	High to very high	MH
A-6		Good stability when compacted in unsoaked condition. 'Fair to poor subgrade material	1.28 – 1.76	2 – 15	1.39 – 5.55	Very poor	High	Medium to high	CL
A-7	A-7-5	Good stability when properly compact & in unsoaked conditions; poor subgrade material when wet.	1.28 – 1.76	2 – 15	1.39 – 5.55	Very poor	Very high	Medium	CL, OL, CH, OH
	A-7-6								

6.1.7 Subgrade Soil Strength

The factor on which the strength characteristics of soil depend are :

(i) soil type

(ii) moisture content

(iii) dry density

(iv) internal structural of the soil, and

(v) the type and mode of stress application

The problem of predicting the stress-strain relationship of soil is difficult, because of the diversity in the soil types and the non-homogeneous nature of the soil under the foundations. Generally the highway engineer is interested in the stability or the resistance to deformation of the soil under the stress applications.

In a soil mass, the deformation is largely due to slippage between soil particles. Hence the *shearing resistance* in soil represents the strength. The sliding mechanism in soils is complicated as the shear deformation cause reorientation of particles resulting in changes in volume, valence bond between particles, thickness and other properties of adsorbed layer of water.

Though many theories of failures of materials are known, *Mohr's theory* is the most useful one for soils. The basis of his theory is that a material fails when the shearing stress on the failure plane is definite function of the normal stress acting on that plane and that failure occurs by slippage only. Shearing resistance in a soil mass is commonly attributed to internal *friction* and *cohesion* parameters of the soil. For majority of soils shearing resistance is made up of both friction and cohesion. For these soils the shearing resistance on any plane is given by *Coulomb's* empirical law :

$$S_r = C + \sigma \tan \phi \qquad (6.2)$$

Here C is cohesion per unit area, ϕ is the angle of internal friction and σ is the normal stress. The value of ϕ depends on the dry density of the soil, grain size distribution, shape and texture of soil strength hence depends on the value of ϕ and the normal pressure on sliding plane, σ. Cohesion C is the resistance of soil grains to displacement by bond developed at the surface of contact by very fine grained soils as result of intermolecular and electrochemical forces of attraction. Cohesion may be said to include, in simple terms, both the *true cohesion* which is due to intermolecular attraction and the *apparent cohesion* which is due to surface tension effects of the held water. The value of C depends on the type of clay mineral, its size, the surface charges, the proportion of the clay and the water content.

The stability of a soil or its strength property is often determined from its stress-deformation characteristics. This depends very much on the type of stress application, its intensity and rate of application. In case of static stresses the period of stress application and the intensity of stress application have significant effect on soils which show *viscoelastic behaviour*. In case of repeated application of stresses, the frequency of loading cycle, the magnitude of stress and the number of repetitions have influence.

6.1.8 Evaluation of Soil Strength

The tests used to evaluate the strength properties of soils may be broadly divided into three groups :

(i) Shear tests,

(ii) Bearing tests, and

(iii) Penetration tests

There are number of test methods in each group.

Shear tests are usually carried out on relatively small soil samples in the laboratory. In order to find the strength properties of a soil, a number of representative samples from different locations are tested. Some of the commonly known shear tests are direct shear test, triaxial compression test and unconfined compression test. Vane shear tests may be carried out either on a soil sample or in-situ soil in the field.

Bearing tests are loading tests carried out on subgrade soils in situ with a load bearing area. The results of the bearing tests are influenced by the variations in the soil properties within the stressed soil mass underneath and hence the overall stability of the part of the soil mass stressed could be studied.

Penetration tests may be considered as small scale bearing tests in which the size of the loaded area is relatively much smaller and ratio of the penetration to size of loaded area is much greater than the ratios in bearing tests. The penetration tests are carried out in the field or in the laboratory. The *California Bearing Ratio* test and cone penetration tests are commonly known penetration tests.

There are number of factors which affect the results of the strength tests as mentioned below :

(i) Factors which are primarily associated with the actual tests such as size and shape of the specimen, method of loading, rate of loading and drainage conditions.

(ii) Factors which are associated with the soil such as soil type dry density, moisture content, permeability structure and other properties of the soil

Some of the commonly adopted tests for evaluating soil strength characteristics are briefly discussed here.

Direct shear test

This is one of the oldest of shear tests. The direct shear apparatus consists essentially of a box divided horizontally into two halves. One half is kept fixed and the other half is free to move horizontally. A vertical load is applied and the horizontal pull is caused to produce a certain rate of horizontal displacement. The vertical and horizontal movements are measured by dial gauges and the horizontal force is noted from the proving ring dial. The maximum horizontal force is measured for different values of normal load. The values of maximum shear stress applied for the different vertical stresses are computed and plotted as shown in Figure 6.5. The values of cohesion (C) and angle of internal friction (ϕ) are found using Coulomb's equation for shear stress (Equation 6.2).

There are a number of limitations of this test. The failure plane being predetermined horizontal plane, need not necessarily represent the *imminent plane* of failure. The shearing stress and strain along this horizontal failure plane is seldom uniform. The area of cross section of specimen decreases with displacement. The flow of water to or from the soil specimen can not be easily controlled or measured. It is also not practicable to measure pore water pressure during the test. Carrying out an undrained (quick) test on a sandy soil is rather impracticable in the shear box.

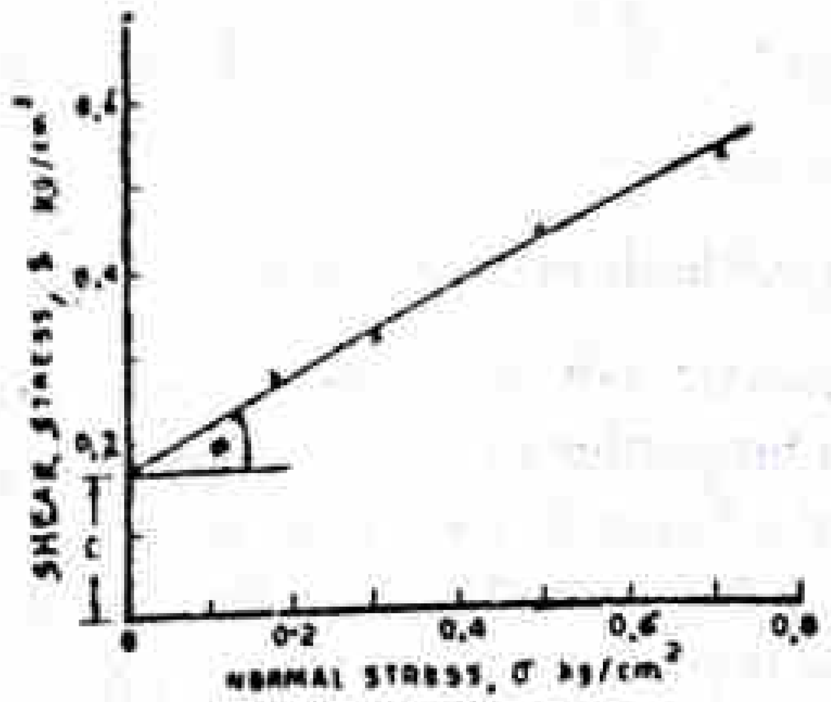


Fig. 6.5 Direct Shear Test Results

Triaxial compression test

The object of triaxial compression test is to determine the shear strength of soil under *lateral confinement*. An attempt is made to simulate the confining pressure observed in a loaded soil mass. In the test set up, it is possible to provide uniform fluid-confining pressure only. Cylindrical specimen of height to diameter ratio 2 is inserted in a thin membrane, placed in a triaxial cell and the required lateral pressure is applied. The normal load is applied through vertical piston by means of a strain controlled machine and the maximum vertical load at failure is recorded. The specimens are usually subjected to a constant strain rate of 1.25 mm per minute. Usually the lateral pressure, σ_3 is maintained constant and the vertical pressure, σ_1 is increased until the specimen fails. In some studies the volume of the specimen is maintained constant by adjusting the lateral pressure, σ_3 during application of vertical stress, σ_1. The specimen may either fail by shearing or in the case of saturated clayey soils by *bulging*. The deviator stress, σ_d under which the specimen fails is $(\sigma_1 - \sigma_3)$. (See Fig. 6.6). The various values of normal stress, σ_1 and hence deviator stress, σ_d corresponding to the different values of lateral pressure σ_3 are obtained from the triaxial tests.

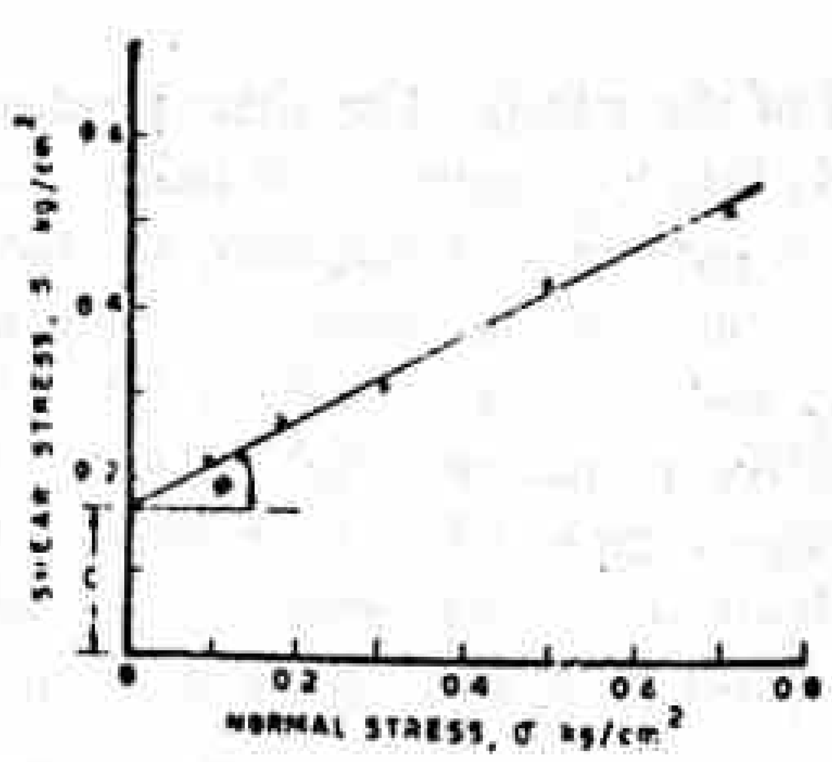


Fig. 6.6 Stresses in Triaxial Test

The tests are carried out at different lateral pressures preferably with atleast three lateral pressures. For the pavement design, lateral pressure of 0, 0.75 and 1.5 kg/cm^2 are considered desirable. The values of confining pressure and total vertical pressure at failure are plotted and semicircles passing through these points are drawn as shown in

Fig. 6.7. A common line tangential to the circles is drawn, representing the *Mohr rupture envelope*. The intercept of this line with Y-axis represents the cohesion C and the inclination with X-axis represents the angle of internal friction ϕ of the soil.

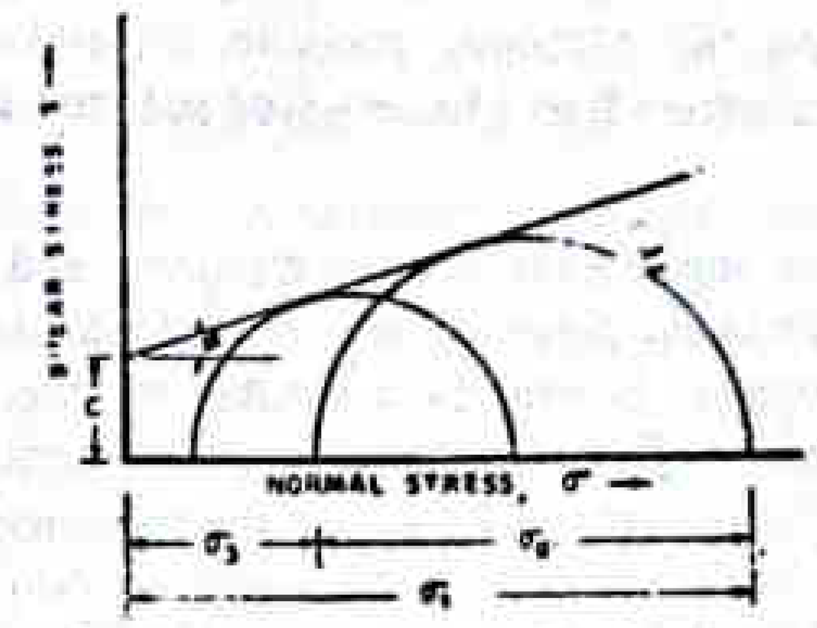


Fig. 6.7 Triaxial Test Results

Correction for Area of Cross Section

In is necessary to correct the stress value for the increased area of cross section due to loading. The corrected value of deviator stress is found with the assumption that the volume of the specimen remains constant and the area of cross section of the specimen is uniformly increased from the original value of A_o to A_1 due to load P_1. Volume $A_o l_o = A_1 l_1 = A_1 (l_o - \Delta)$. In that case the deviator stress σ_d is given by :

$$\sigma_d = \frac{P_1}{A_1} = \frac{P_1 . l_1}{A_o l_o} = \frac{P_1}{A_o} = \left(\frac{l_1 - \Delta}{l_o}\right)$$

i.e. $$\sigma_d = \frac{P_1}{A_o}(1 - \delta) \qquad (6.3)$$

Here Δ = total deformation of the specimen = $(l_o - l_1)$

δ = unit strain = Δ / l_o

A_o = original area of cross section

P_1 = applied load

σ_d = deviator stress.

After calculating the corrected value of σ_d, σ_1 is calculated from the relation, $\sigma_1 = \sigma_d + \sigma_3$.

Basic Types of Triaxial Tests

The behaviour of the soil specimen during testing is influenced by the condition of drainage allowed during the test.

Drained tests or slow tests are those in which the drainage of specimen is permitted during the application of both stresses, σ_3 and σ_d. The specimen is allowed to fully consolidate under the confining pressure, σ_3 and then the deviator stress, σ_d is applied and increased so slowly that no significant pore pressure is built up while the specimen is under the test.

In consolidated-undrained test or consolidated-quick test, complete consolidation is allowed under the confining pressure σ_3 before applying deviator stress. No drainage is permitted when the deviator stress is applied.

In undrained or quick test, no drainage is allowed at any stage. The drainage is prevented before applying the confining pressure, σ_3 and during the application of deviator stress until the specimen fails. Excess pore pressure commonly exist throughout the test.

In pavements the load applied are mostly *transient* and during the loading cycle drainage can not take place in the subgrade soil. In order to simulate consolidation of the subgrade under the pavement, it may be desirable to consolidate the sample to an equivalent confining pressure before the application of the load and thus the consolidated quick test is justified for the pavement design. If the pavement is designed for sustained loading conditions as for the parked or stationary vehicle, then the possibility of drainage in the subgrade during the loaded period may be considered.

Interpretation of Results of Triaxial Test

Besides values of C and ϕ the behaviour of the soil under the loading conditions is judged from the stress-strain relationship of the soil as shown in Figure 6.8. These plots seldom indicate a straight portion even at the initial stages of stressing. Hence it is not possible to interpret the strength of soil in terms of modulus of elasticity as done in the case of various structural materials. In flexible pavement design the *modulus of deformation* of the soil is used. The modulus of deformation is the ratio of stress to strain at an arbitrary point on the stress-strain curve. This point is decided based on the anticipated stress in the subgrade under the pavement or an allowable value of deformation. As an example, if the anticipated stress under a flexible pavement is p kg/cm^2, then the modulus of deformation is equal to p/δ where δ is deformation corresponding to a stress of p kg/cm^2, in the triaxial test. Now the value of the modulus of deformation at any stress level will depend on the confining pressure σ_3 applied. Hence it becomes necessary to decide the value of confining pressure for the triaxial test. However the term modulus of elasticity is also used instead of modulus of deformation for usual computations.

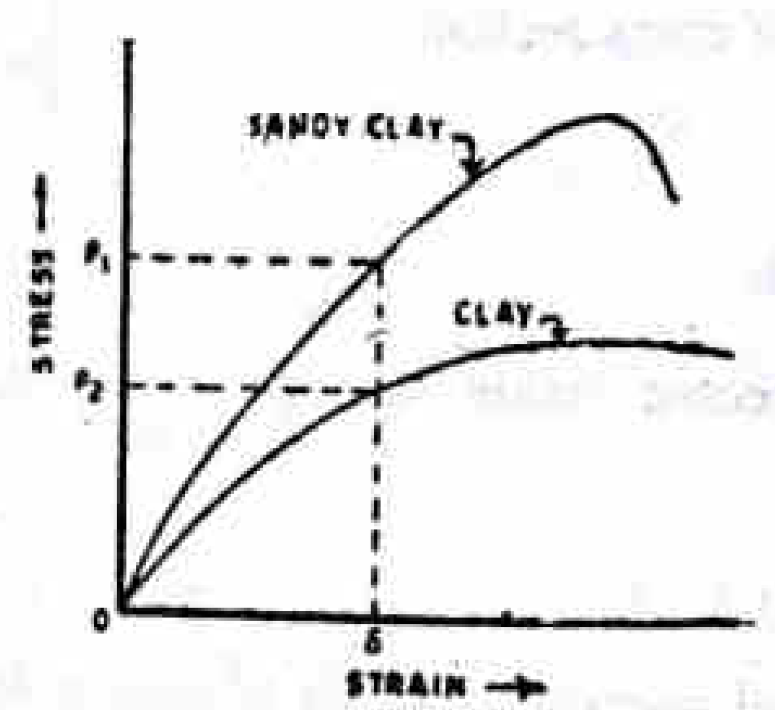


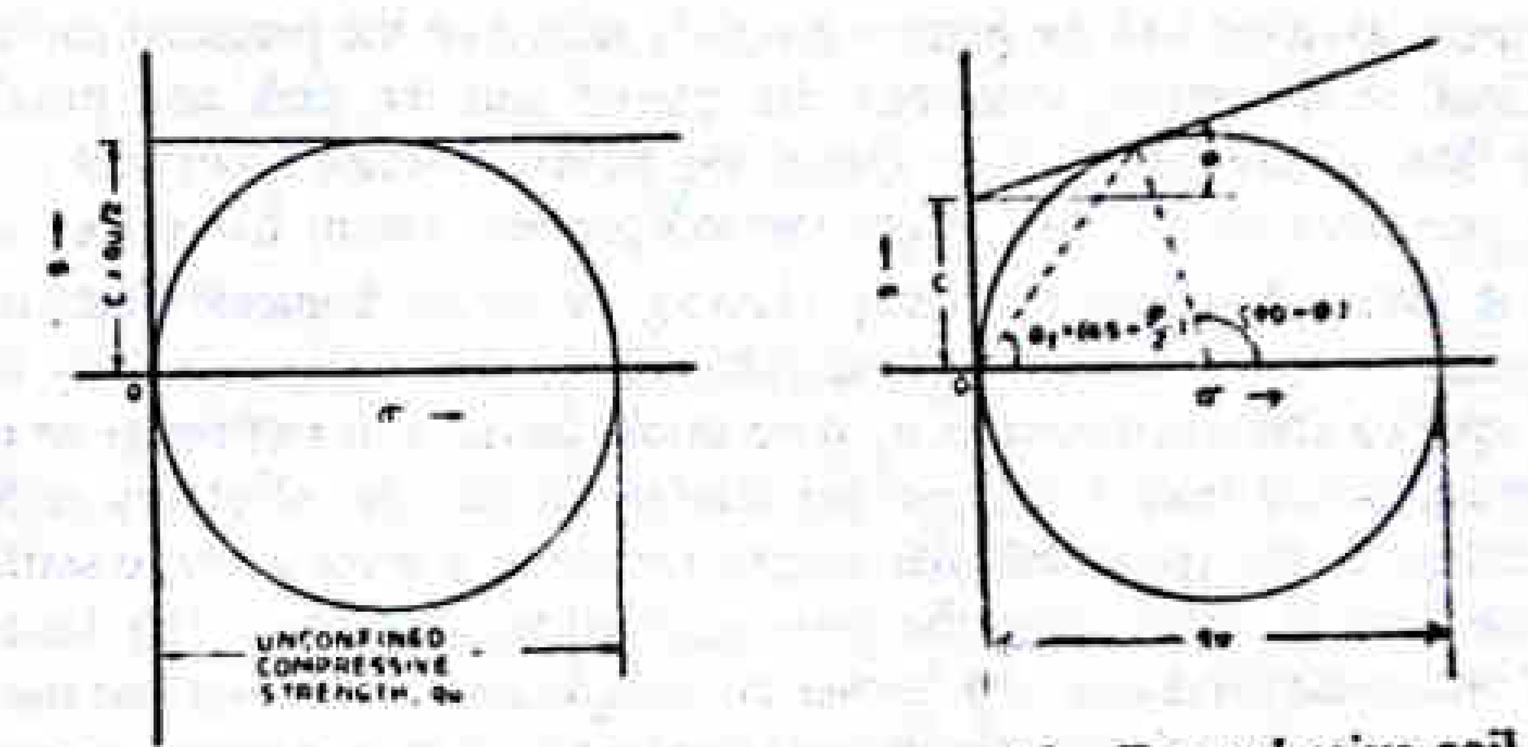
Fig. 6.8 Stress-Strain Relationship of soils

Unconfined compression test

The unconfined compression test may be considered as a special case of triaxial compression test when the confining pressure is zero and the axial compressive stress only is applied to the cylindrical specimen. The stress may be applied and the

deformation and load readings are noted until the specimen fails. The area of cross section of specimen for various strains may be corrected assuming that the volume of the specimen remains constant and that the specimen retains cylindrical cross sectional areas as explained under triaxial test. The maximum axial strain is noted. In clays when plastic failure takes place, no definite value of maximum or peak stress could be noticed; in such cases the stress at 20 percent strain is some time adopted.

The Mohr circle of rupture for an unconfined compression test passes through the origin. It is not possible to draw the Mohr rupture envelope from a single circle, and so the values of C and ϕ cannot be determined as such. In case of quick or undrained test carried out on saturated clays, the value of ϕ may be assumed as zero and hence the Mohr envelope will be a horizontal line touching the circle with the unconfined compressive strength q_u as diameter as shown in Fig. 6.9(a). Hence cohesion is half the unconfined compressive strength in this case i.e., $C = q_u/2$.



(a) For saturated clay where $\phi = 0$ **(b) For cohesive soil**

Fig. 6.9 Unconfined Compression Test Results

The failure plane of an unconfined compression or triaxial specimen (when a definite failure plane exists) makes an angle $\theta_f = (45 + \phi/2)$ with major principal plane. Hence if the angle made by the failure plane in the specimen with the horizontal, θ_f, is noted, ϕ may be calculated. The value of C also can be calculated from the geometrical relationship of the Mohr diagram. The values of C and ϕ may be very easily found graphically as shown in Fig. 6.9 (b).

The unconfined compression or the simple compression is suitable to test cohesive soils and all materials having high values of cohesion or bond strength such as soil-cement etc.

Plate bearing test

The plate bearing tests is used to evaluate the supporting power of subgrade for use in pavement design by using relatively large diameter plates. The plate bearing test was originally devised to find the *modulus of subgrade reaction* in the Westergaard's analysis for wheel load stress in cement concrete pavements.

The test set up consists of a set of plates of diameter 75, 60, 45 and 30 cm, a loading device consisting of jack and proving ring arrangement and a reaction frame against which the jack can give a thrust to the plate. A datum frame resting far from the loaded area and dial gauges from this frame are used to measure the settlement of the loaded plate. The loading arrangement is shown in Fig. 6.10.

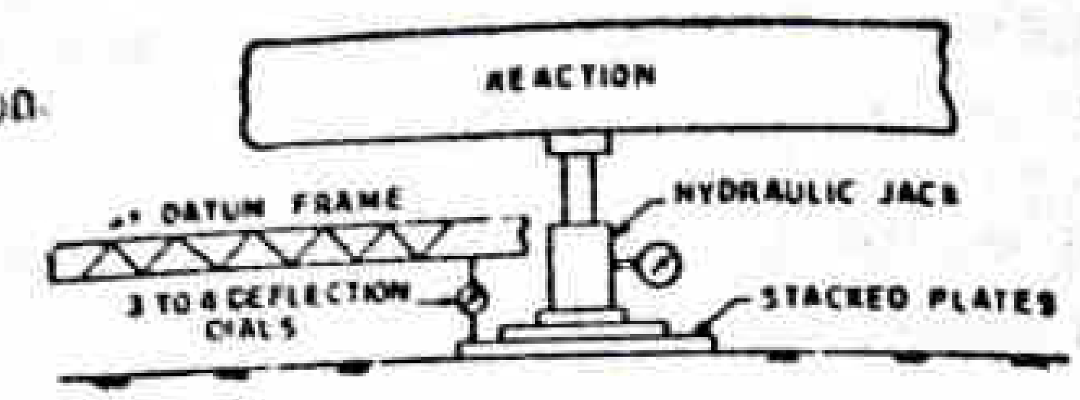


Fig. 6.10 Plate Bearing Test Set Up

Modulus of Subgrade Reaction

Modulus of subgrade reaction K may be defined as the pressure sustained per unit deformation of subgrade at specified deformation or pressure level, using specified plate size. The standard plate size for finding K-value is 75 cm diameter. But in some tests a smaller plate of 30 cm diameter is also used.

The test site is levelled and the plate is properly seated on the prepared surface. The stiffening plates of decreasing diameters are placed and the jack and proving ring assembly are fitted to provide reaction against the frame. Three or four dial gauges are fixed on the periphery of the plate, from the independent datum frame for measuring settlements. A seating load of 0.07 kg/cm^2 (320 kg for 75 cm diameter plate) is applied and released after a few seconds. A load sufficient to cause approximately 0.25 mm settlement is applied and when there is no perceptible increase in settlement or when the rate of settlement is less than 0.025 mm per minute (in the case of clayey soils or wet soils), the readings of the settlement dial gauges are noted and the average settlement is found, and the load is noted from the proving ring dial reading. The load is then increased till settlement increases to a further amount of about 0.25 mm and the average settlement and load are found. The procedure is repeated till the settlement reaches 0.175 cm. A graph is plotted with mean settlement versus mean bearing pressure as shown in Fig. 6.11. The pressure p corresponding to a settlement of 0.125 cm is read and the K-value is calculated by the reaction,

$$K = \frac{p}{0.125} \text{ kg/cm}^2\text{/cm (or kg/cm}^3\text{)} \qquad (6.4)$$

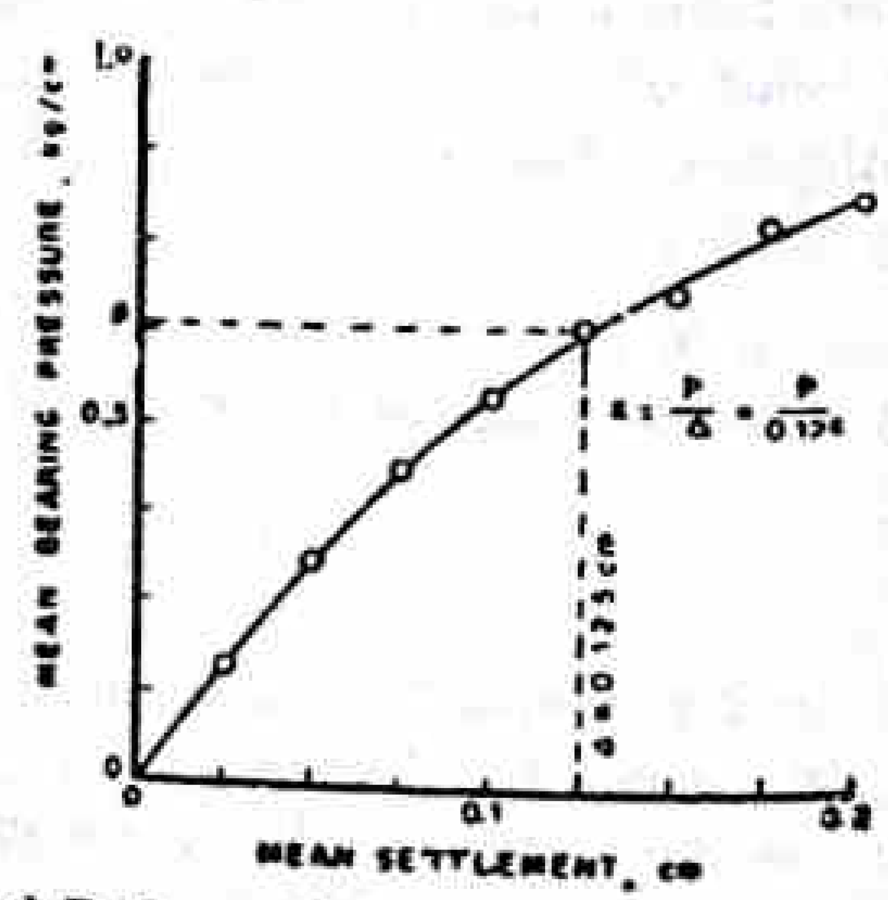


Fig. 6.11 Load-Deformation Curve from Plate Bearing Test

Allowance for Worst Subgrade Moisture

The moisture content at the time of carrying out plate bearing tests may seldom represent the worst moisture condition likely to occur at the test site. It may not be practicable always to carry out the test at such a condition. In such cases the value of modulus of subgrade reaction K is found for the prevailing moisture content and the value so obtained may be modified for soaked condition.

After the plate bearing test, two consolidation test specimens are prepared. One specimen is tested as such without soaking by applying increments of pressure and the pressure deformation curve is plotted as shown in Fig. 6.12. The deformation δ of a sample corresponding to a pressure p kg/cm^2 required in the plate bearing test to cause a deformation of 0.125 cm is noted. Then the other specimen is soaked and consolidation test is carried out; the pressure p_s required to produce the same deformation δ is noted, in the soaked test. (Refer Fig. 6.12). The modulus of subgrade reaction K_s for the soaked condition is then calculated from the relation

$$K_s = K \frac{p_s}{p} \qquad (6.5)$$

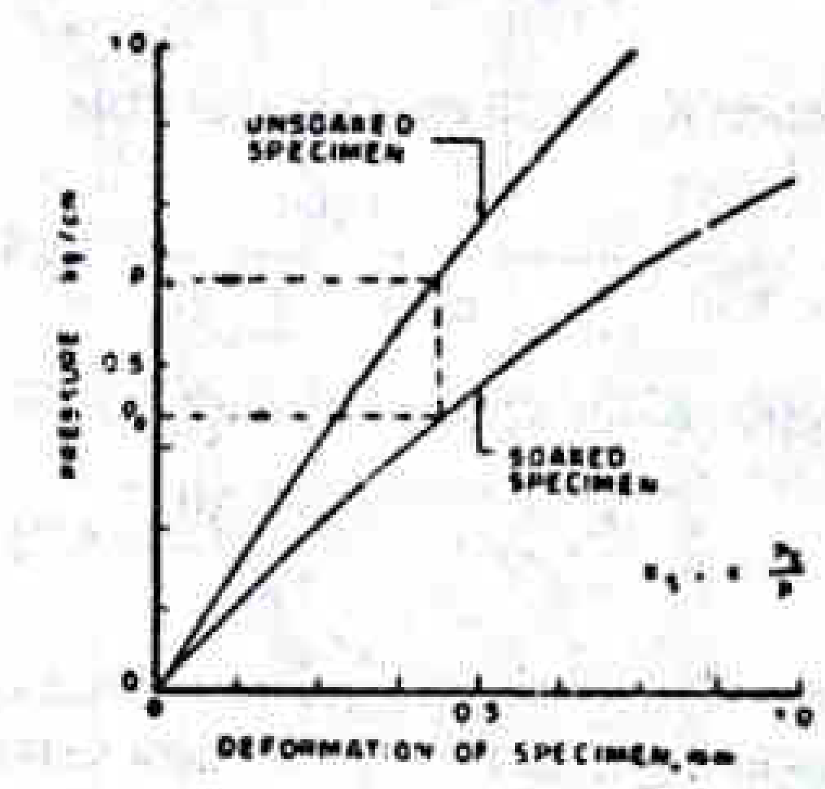


Fig. 6.12 Correction for Soaking in Plate Bearing Test

Correction for Small Plate Size

In some cases the load capacity may not be adequate to cause 75 cm diameter plate to settle 0.175 cm. In such case, a plate of smaller diameter (say 30 cm) may be used. Then K-value should be found by applying a suitable correction for plate size.

Assuming the subgrade to be an elastic medium with modulus of elasticity E (kg/cm^2) the theoretical relationship of deformation, Δ (cm) under a rigid plate of radius a (cm) is given by :

$$\Delta = 1.18 \frac{pa}{E} \qquad (6.6)$$

$$K = \frac{p}{\Delta} = \frac{p.E}{1.18\,p\,a} = \frac{E}{1.18\,a}$$

If the value of E is taken as constant for a soil then K is inversely proportional to *a* or Ka is constant i.e., Ka = $K_1 a_1$ or K = $K_1 a_1$/a. Hence if the test is carried out with a smaller plate of radius a_1 and the modulus of subgrade reaction K_1 is found, then the corrected value of modulus of subgrade reaction K for standard plate of radius a, is obtained from the relationship

$$K = K_1 \frac{a_1}{a} \qquad (6.7)$$

Example 6.3

A plate load test was conducted on a soaked subgrade during monsoon season using a plate diameter of 30 cm. The load values corresponding to the mean settlement dial readings are given below. Determine the modulus of subgrade reaction for the standard plate.

Mean settlement values, mm	0.0	0.24	0.52	0.76	1.02	1.23	1.53	1.76
Load values kg	0.0	460	900	1180	1360	1480	1590	1640

Solution

The load-settlement curve is plotted on a graph paper (similar to the one shown in Fig. 6.11) and the load value p_1 corresponding mean settlement value of $\Delta = 0.125$ cm is determined = 1490 kg.

$$\text{Unit load } p_1 = \frac{1490}{\pi 15^2} \text{ kg/cm}^2$$

Modulus of subgrade reaction K_1 for 30 cm diameter plate

$$= \frac{p_1}{\Delta} = \frac{1490}{\pi . 15^2 \times 0.125} = 16.86 \text{ kg/cm}^3$$

Modulus of subgrade reaction K for standard plate of dia. 70 cm.

$$= \frac{K_1\, a_1}{a} = \frac{16.86 \times 30}{75} = 6.75 \text{ kg/cm}^3$$

(Note : As the plate load test was conducted under soaked condition during monsoon season, there is no need to apply correction for subsequent soaking).

California Bearing Ratio (CBR) test

This is a penetration test developed by the California Division of Highways, as a method for evaluating the stability of soil subgrade and other flexible pavement materials. The test results have been correlated with flexible pavement thickness requirements for highways and air fields. The CBR test may be conducted in the laboratory on a prepared specimen in a mould or in-situ in the field.

The laboratory CBR apparatus consists of a mould 150 mm diameter with a base plate and a collar, a loading frame with the cylindrical plunger of 50 mm diameter and dial gauges for measuring the expansion on soaking and the penetration values, refer Fig. 6.13.

Briefly the penetration test consists of causing a cylindrical plunger of 50 mm diameter to penetrate a pavement component material at 1.25 mm/minute. The load values to cause 2.5 mm and 5.0 mm penetration are recorded. These loads are expressed as percentages of standard load values at respective deformation levels to obtain CBR value. The standard load values obtained from the average of a large number of tests on crushed stones are 1370 and 2055 kg (70 and 105 kg/cm^2) respectively at 2.5 and 5.0 mm penetration.

The specimen in the mould is subjected to four days soaking and the swelling and water absorption values are noted. The surcharge weight is placed on the top of the

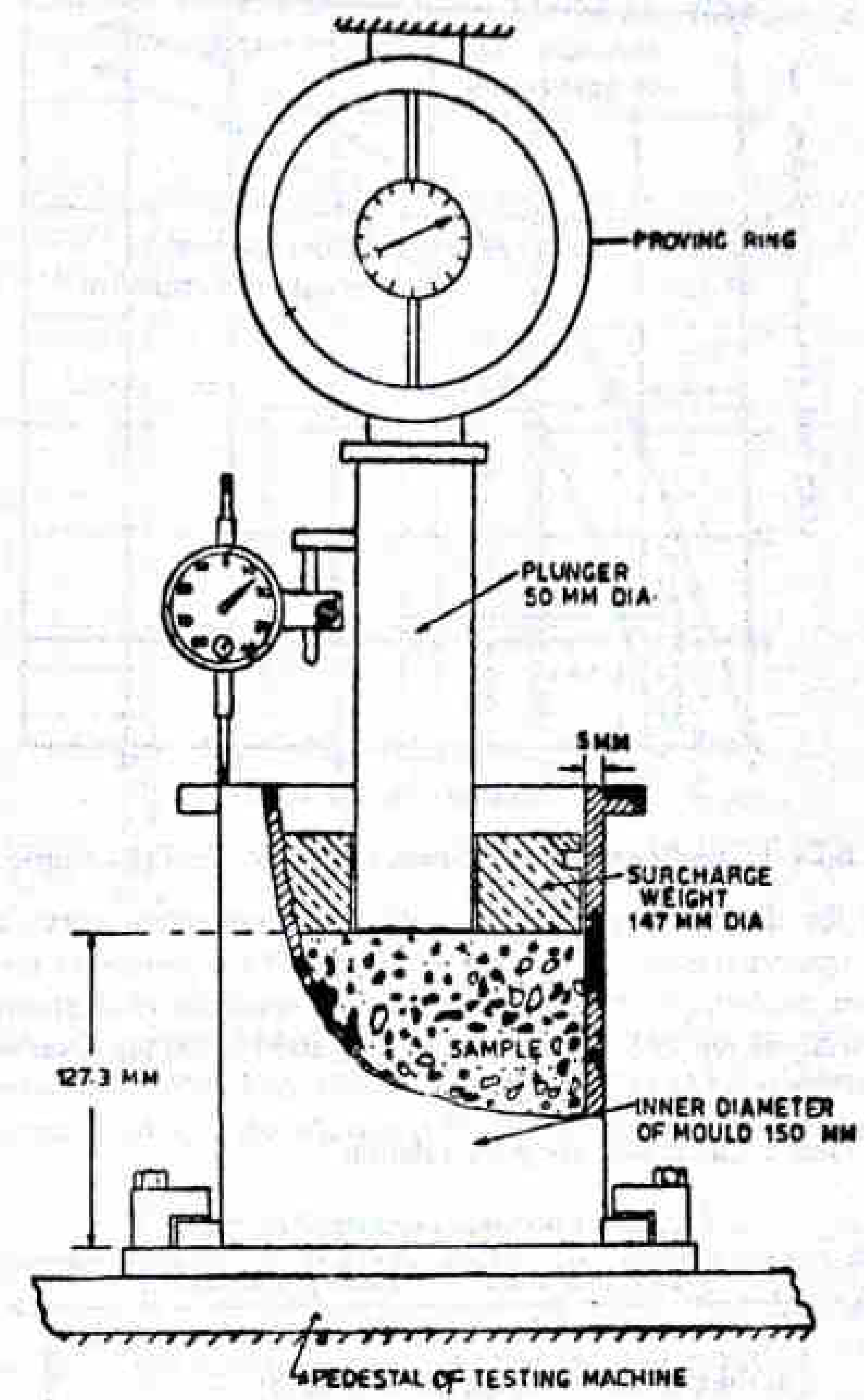


Fig. 6.13 CBR Test Set up

specimen in the mould and the assembly is placed under the plunger of the loading frame as shown in Fig. 6.13. The load values are noted corresponding to penetration values of 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10.0 and 12.5 mm. The load penetration graph is plotted as shown in Fig. 6.14. Alternatively the load values may be converted to pressure values and plotted against the penetration values.

Two typical types of curves may be obtained as shown in Fig. 6.14. The normal curve is with convexity upwards as for Specimen no.1 and the loads corresponding to 2.5 and 5.0 mm penetration values are noted. Some times a curve with initial upward concavity is obtained, indicating the necessity of correction as for Specimen no. 2. In this case the corrected origin is established by drawing a tangent AC from the steepest point A on the curve. The load values corresponding to 2.5 and 5.0 mm penetration values from the corrected origin C are noted.

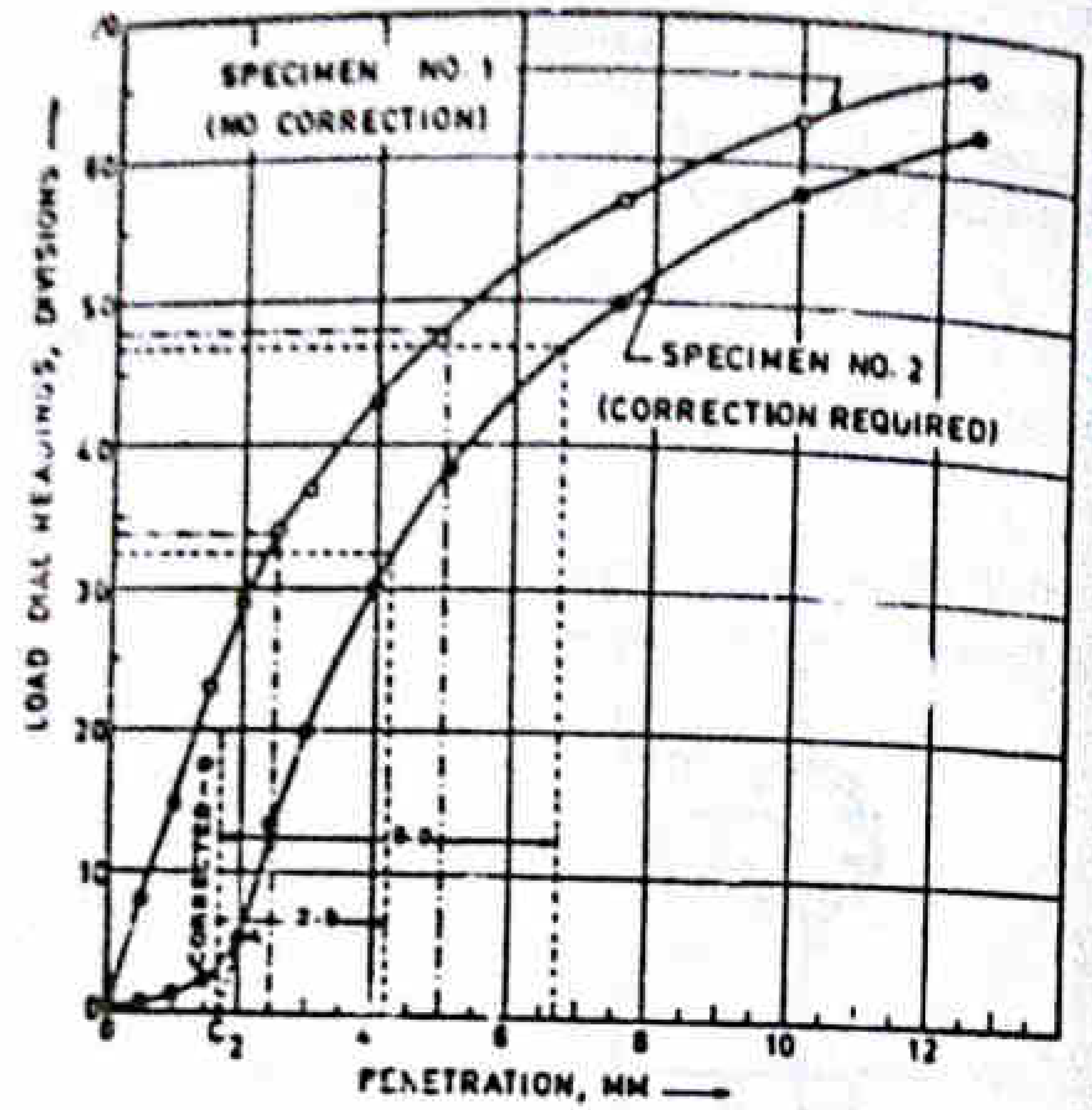


Fig. 6.14 Load-Penetration Curves in C.B.R. Test (Example 6.4)

The causes for the initial concavity of the load-penetration curve calling for the correction in origin are due to : (i) the bottom surface of the plunger or the top surface of the soil specimen not being truly horizontal, with the result the plunger surface not being in full contact with the top of the specimen initially and (ii) the top layer of the specimen being too soft or irregular.

The C.B.R. value is calculated using the relation :

$$\text{CBR, \%} = \frac{\left[\begin{array}{l}\text{Load (or pressure) sustained by the} \\ \text{specimen at 2.5 or 5.0 mm penetration}\end{array}\right]}{\left[\begin{array}{l}\text{Load (or pressure) sustained by standard aggregates} \\ \text{at the corresponding penetration level}\end{array}\right]} \times 100 \qquad (6.8)$$

Normally the CBR value at 2.5 mm penetration which is higher than that at 5.0 mm is reported as the CBR value of the material. However, if the CBR value obtained from the test at 5.0 mm penetration is higher than that at 2.5 mm, then the test is to be repeated for checking. If the check test again gives similar results, the higher value obtained at 5.0 mm penetration is reported as the CBR value. The average CBR value of three test specimens is reported to the first decimal place, as the CBR value of the material. If the variation in CBR value between the three specimens is more than the prescribed limits, tests should be repeated on additional three samples and the average CBR value of six specimens is accepted.

The CBR test is essentially an arbitrary strength test and hence can not be used to evaluate the soil properties like cohesion or angle of internal friction or shearing resistance. Unless the test procedure is strictly followed, dependable results cannot be obtained. Presence of coarse grained particles would result in poor reproducibility of results. Material passing 20 mm sieve is only used in the test. The field CBR test is carried out using in-situ penetration test.

The test is meant for soils and is also carried out on sub-base and granular base course materials. The CBR test values are made use of in an empirical method of flexible pavement design as given in Chapter 7.

Example 6.4

The load penetration values of CBR tests conducted on two specimens of a soil sample are given below. Determine the CBR value of the soil if 100 division of the load dial represents 190 kg load in the calibration chart of the proving ring.

Penetration of plunger, mm	Load dial readings, divisions	
	Specimen No 1	Specimen No 2
0.0	0	0
0.5	8	0.5
1.0	15	1.5
1.5	23	2.5
2.0	29	6.0
2.5	34	13
3.0	37	20
4.0	43	30
5.0	48	38
7.5	57	50
10.0	63	58
12.5	67	63

Solution

The penetration values are plotted against the load dial reading as shown in Fig. 6.14. (Instead, the load dial readings may also be converted either to load values in kg or load per unit area of cross section of the plunger in kg/cm^2 and plotted on the Y-axis.)

Specimen no. 1

The load penetration curve for specimen no.1 is consistently convex throughout and needs no correction.

Load dial reading at 2.5 mm penetration = 34 divisions (Fig. 6.14)

$$\text{Load at 2.5 mm penetration} = 34 \times \frac{190}{100} = 64.6 \text{ kg}$$

$$\text{CBR value at 2.5 mm penetration} = \frac{64.6 \times 100}{1370} = 4.7\%$$

$$\text{CBR value at 5.0 mm penetration} = \frac{48 \times 190 \times 100}{100 \times 2055} = 4.4\%$$

$$\therefore \text{CBR value of Specimen no.1} = 4.7\%$$

Specimen no.2

As the curve has an initial concavity, correction is required. A tangent AC is drawn from the steepest portion A of the curve to intersect the X-axis at C, which is the corrected origin for this specimen. The penetration values are measured from this corrected origin C, as shown in Fig. 6.14.

$$\text{CBR value at 2.5 mm penetration} = \frac{32.5\times190\times100}{100\times1370}$$

$$= 4.5\%$$

$$\text{CBR value at 5.0 mm penetration} = \frac{47\times190}{2055} = 4.3\%$$

$$\text{CBR value of Specimen no.2} = 4.5\%$$

$$\text{Therefore mean CBR value of the soil sample} = \frac{4.7+4.5}{2}$$

$$= 4.6\%$$

Stabilometer Test

Hveem's Stabilometer test is conducted on subgrade soil at various moisture contents. The stabilometer R-value is determined using Eq. 6.17 as explained at the end of this chapter. The result of this test are used in the Stabilometer method of pavement design (See Art. 7.3.4).

6.2 STONE AGGREGATES

6.2.1 Introduction

Aggregates form the major portion of pavement structure and they form the prime materials used in pavement construction. Aggregates have to bear stresses occurring due to the wheel loads on the pavement and on the surface course they also have to resist wear due to abrasive action of traffic. These are used in pavement construction in cement concrete, bituminous concrete and other bituminous constructions and also as granular base course underlying the superior pavement layers. Therefore the properties of the aggregates are of considerable significance to the highway engineers.

Most of the road aggregates are prepared from natural rock. *Gravel* aggregates are small rounded stones of different sizes which are generally obtained as such from some river beds. Sand is fine aggregate from weathering of rock. The properties of the rock from which the aggregates are formed, depend on the properties of constituent materials and the nature of bond between them. Based on the origin, natural rocks are classified as igneous, sedimentary and metamorphic. Texture is an important factor, affecting the property of the rock and the fragments.

The aggregates are specified based on their grain size, shape, texture and its gradation. Aggregate size is ascertained by sieving through square sieves of successively decreasing sizes. The required aggregate sizes are chosen to fulfil the desired gradation. The gradings for different road making purposes have been specified by various agencies like the A.S.T.M., B.S.I., I.S.I. and the I.R.C.

Based on the strength property, the coarse aggregates may be divided as *hard aggregates and soft aggregates*. Generally for the bearing course of superior pavement types, hard aggregates are preferred to resist the abrading and crushing effects of heavy traffic loads and to resist adverse weather conditions. In the case of low-cost road construction for use in lower layers of pavement structures, soft aggregates can also be used. The soft aggregate include *moorum, kankar, laterite, brick aggregates and slag*. A different set of test specifications are adopted for soft aggregates.

6.2.2 Desirable Properties of Road Aggregates

Strength

The aggregates to be used in road construction should be sufficiently strong to withstand the stresses due to traffic wheel load. The aggregates which are to be used in top layers of the pavements, particularly in the wearing course have to be capable of withstanding high stresses in addition to wear and tear; hence they should possess sufficient strength resistance to crushing.

Hardness

The aggregates used in the surface course are subjected to constant rubbing or *abrasion* due to moving traffic. They should be hard enough to resist the wear due to abrasive action of traffic. Abrasive action may be increased due to the presence of abrasive material like sand between the tyres of moving vehicles and the aggregates exposed at the top surface. This section may be severe in the case of steel tyred vehicles. Heavy wheel loads can also cause deformations on some types of pavement resulting in relative movement of aggregates and rubbing of aggregates with each other within the pavement layer. The mutual rubbing of stones is called *attrition*, which also may cause a little wear in the aggregates; however attrition will be negligible or absent in most of the pavement layers.

Toughness

Aggregates in the pavements are also subjected to impact due to moving wheel loads. Sever impact like hammering is quite common when heavily loaded steel tyred vehicles move on water bound macadam roads where stones protrude out especially after the monsoons. Jumping of the steel tyred wheels from one stone to another at different levels causes severe impact on the stones. The magnitude of impact would increase with the roughness of the load surface, the speed of the vehicle and other vehicular characteristics. The resistance to impact or toughness is hence another desirable property of aggregates.

Durability

The stone used in pavement construction should be durable and should resist disintegration due to the action of weather. The property of the stones to withstand the adverse action of weather may be called *soundness*. The aggregates are subjected to the physical and chemical action of rain and ground water, the impurities there-in and that of atmosphere. Hence it is desirable that the road stones used in the construction should be sound enough to withstand the weathering action.

Shape of Aggregates

The size of the aggregates is first qualified by the size of square sieve opening through which an aggregate may pass, and not by the shape. Aggregates which happen to fall in a particular size range may have rounded, cubical, angular flaky or elongated shape of particles. It is evident that the flaky and elongated particles will have less strength and durability when compared with cubical, angular or rounded particles of the same stone. Hence too flaky and too much elongated aggregates should be avoided as far as possible. Rounded aggregates may be preferred in cement concrete mix due to low specific surface area and better workability for the same proportion of cement paste and same water cement ratio, whereas rounded particles are not preferred in granular base course, WBM construction and bituminous construction as the stability due to interlocking of rounded particles is less. In such constructions angular particles are preferred. The voids present in a compacted mix of coarse, aggregates depends on the shape factors. Highly angular flaky and elongated aggregates have more voids in comparison with rounded aggregates.

Adhesion with Bitumen

The aggregates used in bituminous pavements should have less affinity with water when compared with bituminous materials; otherwise the bituminous coating on the aggregate will be stripped off in presence of water.

6.2.3 Tests for Road Aggregate

In order to decide the suitability of the road stones for use in construction, the following tests are carried out :

(a) Crushing test

(b) Abrasion test

(c) Impact test

(d) Soundness

(e) Shape test

(f) Specific gravity and water absorption test

(g) Bitumen adhesion test

The essential features of these tests are discussed below. Separate tests are available for testing cylindrical stone specimens and coarse aggregates for crushing, abrasion and impact tests. But due to the difficulties of preparing cylindrical stone specimen which need costly core drilling, cutting and polishing equipment, the use of such tests are now limited. Testing of aggregates is easy and simulate the field condition better, as such these are generally preferred.

Aggregate crushing test

The strength of coarse aggregate may be assessed by aggregate crushing test. The aggregate crushing value provides a relative measure of resistance to crushing under gradually applied compressive load. To achieve a high quality of pavement, aggregates possessing high resistance to crushing or low aggregate crushing value are preferred.

The apparatus for the standard test consists of a steel cylinder 15.2 cm diameter with a base plate and a plunger, compression testing machines, cylindrical measure of diameter 11.5 cm and height 18 cm, tamping rod and sieves. The sketch of the test cylinder and accessories is shown in Fig. 6.15.

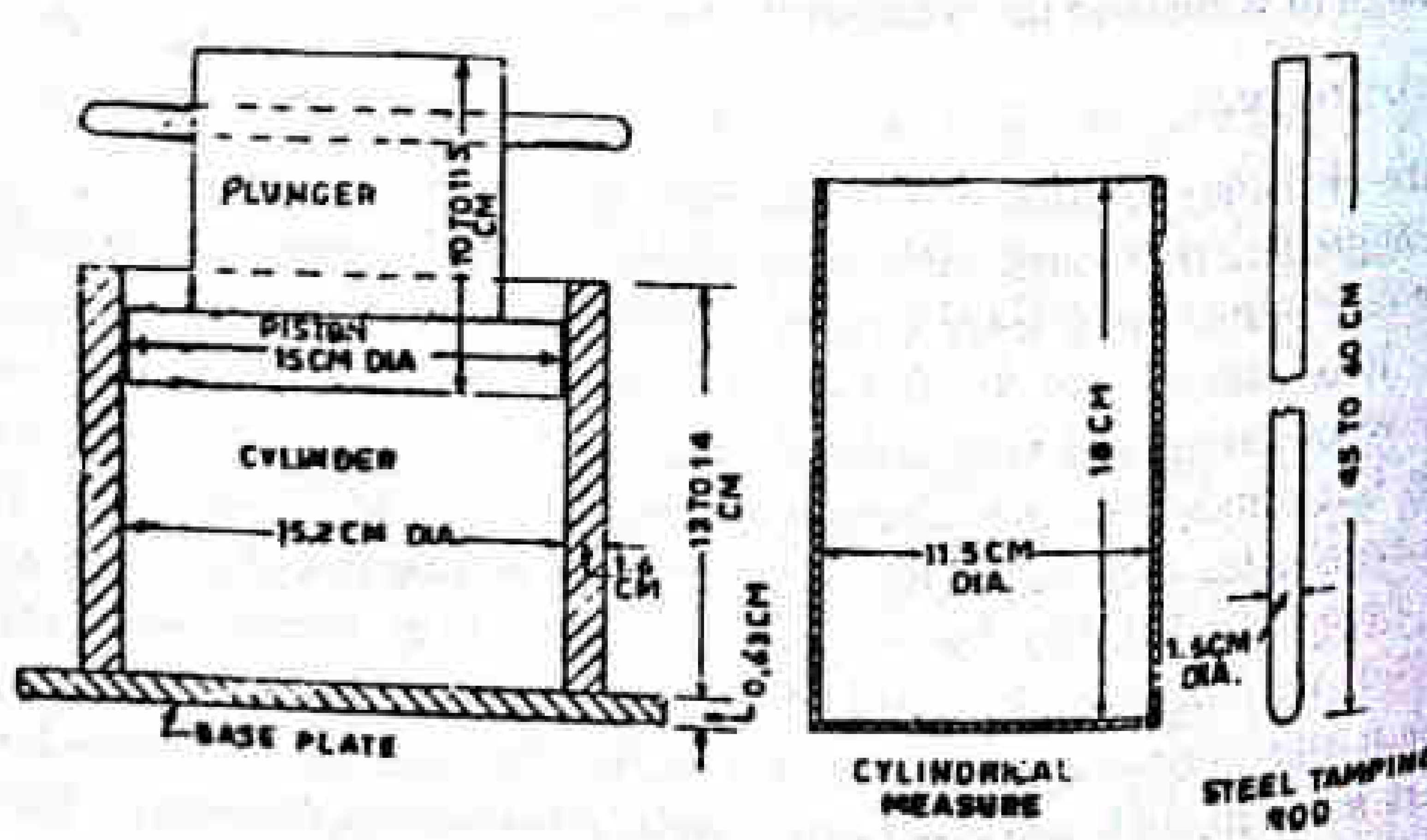


Fig. 6.15 Aggregate Crushing Test Apparatus

Dry aggregate passing 12.5 mm IS sieve and retained on 10 mm sieve is filled in the cylindrical measure in three equal layers, each layer being ramped 25 times by the tamper. The test sample is weighed (equal to W_1 g) and placed in the test cylinder in three equal layers, tamping each layer 25 times. The plunger is placed on the top of specimen and a load of 40 tonnes is applied at a rate of 4 tonnes per minute by the compression machine. The crushed aggregate is removed and sieved on 2.36 mm IS sieve. The crushed material which passes this sieve is weighed equal to W_2 g. The aggregate crushing value is the percentage of the crushed material passing 2.36 mm sieve in terms of original weight of the specimen.

$$\text{Aggregate crushing value} = \frac{100\,W_2}{W_1} \text{ percent}$$

Strong aggregates give low aggregate crushing value. The aggregate crushing value for good quality aggregate to be used in base course shall not exceed 45 percent and the value for surface course shall be less than 30 percent.

Abrasion tests

Due to the movements of traffic the road stones used in the surface course are subjected to wearing action at the top. Hence road stones should be hard enough to resist the abrasion due to the traffic. Abrasion tests are carried out to test the hardness property of stones and to decide whether they are suitable for the different road construction works. The abrasion test on aggregate may be carried out using any one of the following three tests :

(i) Los Angeles abrasion test

(ii) Deval abrasion test

(iii) Dorry abrasion test

However Los Angeles abrasion test is preferred as the test results have been correlated with pavement performance.

Los Angeles Abrasion Test

The principle of Los Angeles abrasion test is to find the percentage wear due to the relative rubbing action between the aggregate and steel balls used as abrasive charge. Pounding action of these balls also exists during the test and hence the resistance to wear and impact is evaluated by this test. The Los Angeles machine consists of a hollow cylinder closed at both ends, having inside diameter 70 cm and length 50 cm and mounted so as to rotate about its horizontal axis. The machine is shown in Fig. 6.16. The abrasive charge consists of cast iron spheres of approximate diameter 4.8 cm and each of weight 390 to 445 g. The number of spheres to be used as abrasive charge and their total weight have been specified based on grading of the aggregate sample. The test has been standardised by the ISI.

The specified weight of aggregate specimen, (5 to 10 kg, depending on gradation) is placed in the machine along with the abrasive charge. The machine is rotated at a speed of 30 to 33 rpm for the specified number of revolutions (500 to 1000 depending on the grading of the specimen). The abraded aggregate is then sieved on 1.7 mm IS sieve, and the weight of powdered aggregate passing this sieve is found. The result of the abrasion test expressed as the percentage wear or the percentage passing 1.7 mm sieve expressed

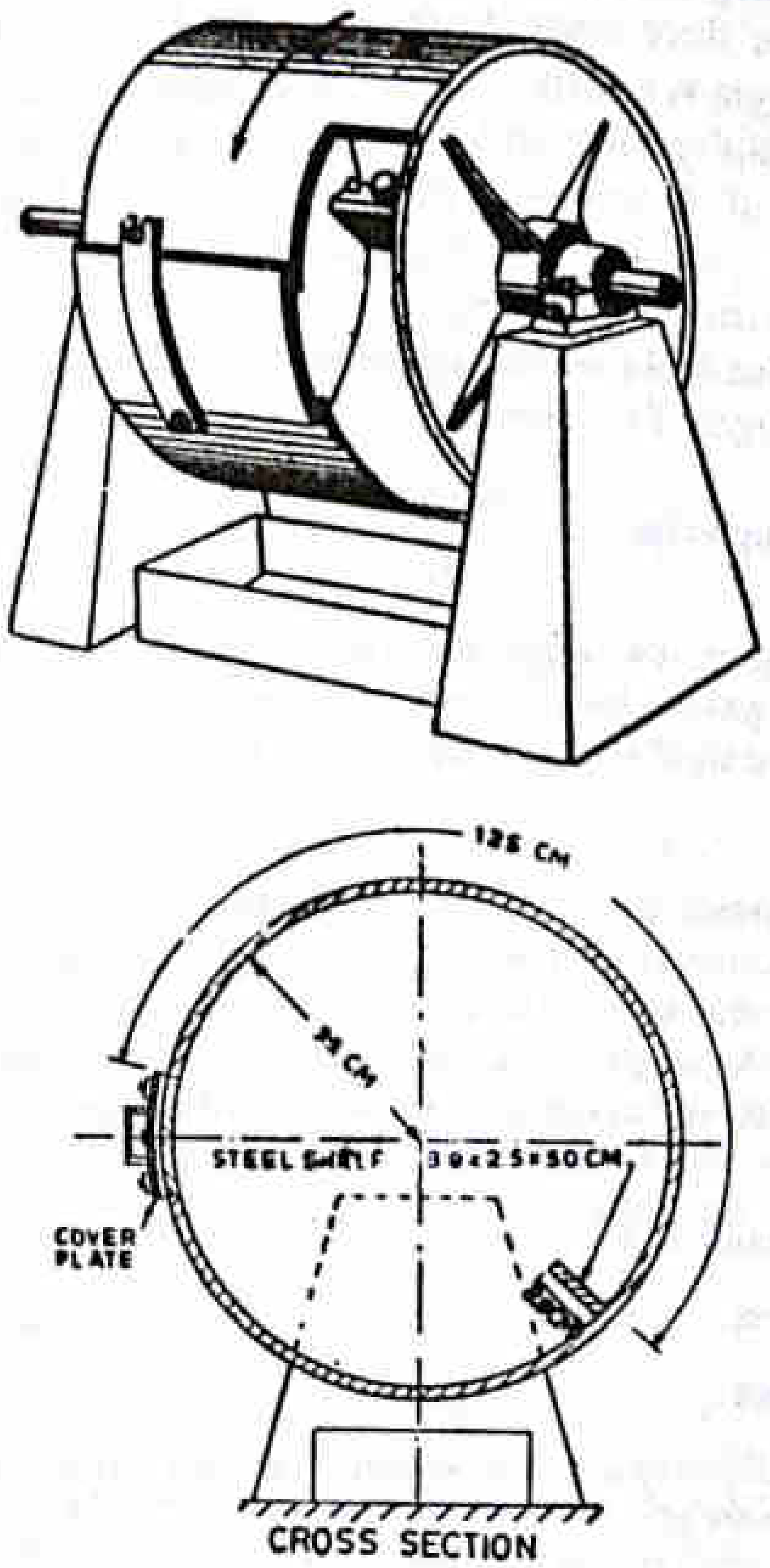


Fig. 6.16 Los Angeles Abrasion Testing Machine

in terms of the original weight of the sample. The Los Angeles abrasion value of good aggregates acceptable for cement concrete, bituminous concrete and other high quality pavement materials should be less than 30 percent. Values up to 50 percent are allowed in base courses like water bound and bituminous macadam. This test is more dependable than other abrasion tests as rubbing and pounding action in the test simulate the field conditions better. Also correlation of Los Angeles abrasion value with field performance and specifications of the test values have been established.

Deval Abrasion Test

The principle of the test is by allowing the sample of aggregate specimen to tumble over in a rattler in the presence of abrasive charge. The Devel machine consists of two hollow cylinder of diameter 20 cm and length 34 cm mounted in such a way that the cylinder rotate about a horizontal axis, but the axis of the cylinders make 30° angle with the horizontal. The schematic sketch of the machine is shown in Fig. 6.17. Specified quantity of dry aggregate specimen (4 to 5.5 kg), of any one of the specified gradings is placed in a cylinder. The abrasive charge consisting of 6 cast iron or steel spheres of

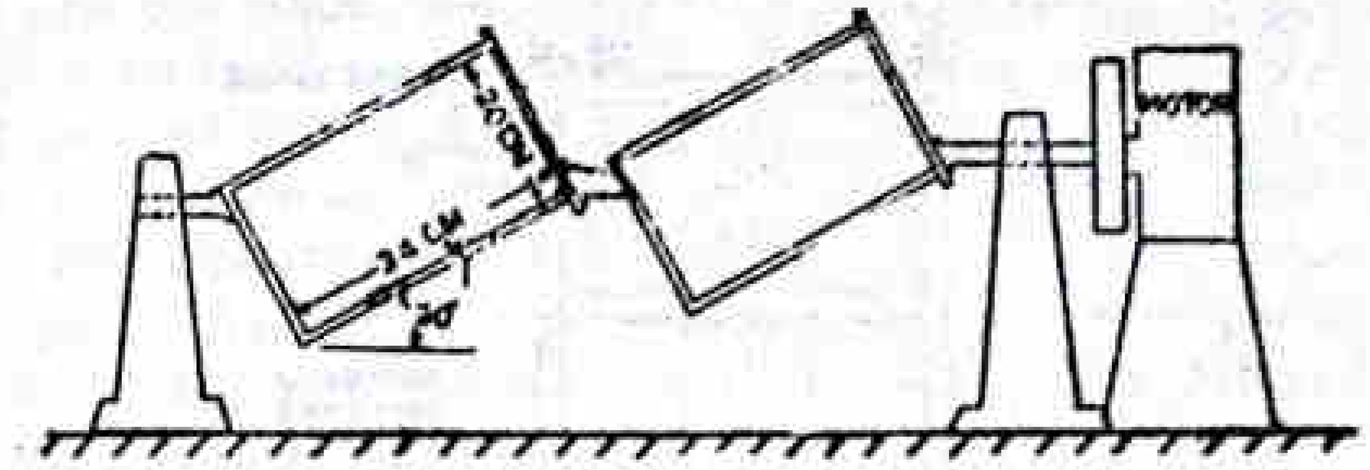


Fig. 6.17 Deval Abrasion Testing Machine

about 4.8 cm diameter and total weight 2500 g is placed. Two tests may be carried out simultaneously using both the cylinders. The machine is rotated at a speed of 30 to 33 rpm. After 10,000 revolutions the material is sieved on 1.7 mm IS sieve. The material passing this sieve is expressed as the percentage of the original weight of the sample and is reported as the abrasion value.

When the test is carried out by Deval machine without using abrasive charges, the test is known as Deval *attrition test*. However this test is not commonly carried out.

Dorry Abrasion Test

The abrasion value of aggregate is also determined using Dorry abrasion testing machine. This is a British method. The machine consists of a flat circular iron disc of 60 cm diameter which is rotated in a horizontal plane at 28 to 30 rpm. Two rectangular trays are kept 26 cm from the centre of the disc to hold the aggregate sample in a specified manner. Abrasive sand is fed through the funnel and the disc is subjected to 500 revolutions. The abrasion value is expressed as the percent loss in weight due to abrasion.

Impact test

A test designed to evaluate the toughness of stone or the resistance of the aggregates to fracture under repeated impacts is called impact test. The aggregate impact test is commonly carried out to evaluate the resistance to impact of aggregates and has been standardised by ISI.

The aggregate impact value indicates a relative measure of resistance of aggregate to impact, which has a different effect than the resistance to gradually increasing compressive stress. The aggregate impact testing machine consists of a metal base and a cylindrical steel cup of internal diameter 10.2 cm and depth 5 cm in which the aggregate specimen is placed. A metal hammer of weight of 13.5-14.0 kg having a free fall from a height 38 cm is arranged to drop through vertical guides. The aggregate impact machine is shown in Fig. 6.18.

Aggregate specimen passing 12.5 mm sieve and retained on 10 mm sieve is filled in the cylindrical measure in 3 layers by tamping each layer by 25 blows. The sample is transferred from the measure to the cup of the aggregate impact testing machine and compacted by tamping 25 times. The hammer is raised to a height of 38 cm above the upper surface of the aggregate in the cup and is allowed to fall freely on the specimen. After subjecting the test specimen to 15 blows, the crushed aggregate is sieved on 2.36 mm sieve. The aggregate impact value is expressed as the percentage of the fine formed in terms of the total weight of the sample.

The aggregate impact value should not normally exceed 30 percent for aggregate to be used in wearing course of pavements. The maximum permissible value is 35% for bituminous macadam and 40% for water bound macadam base courses.

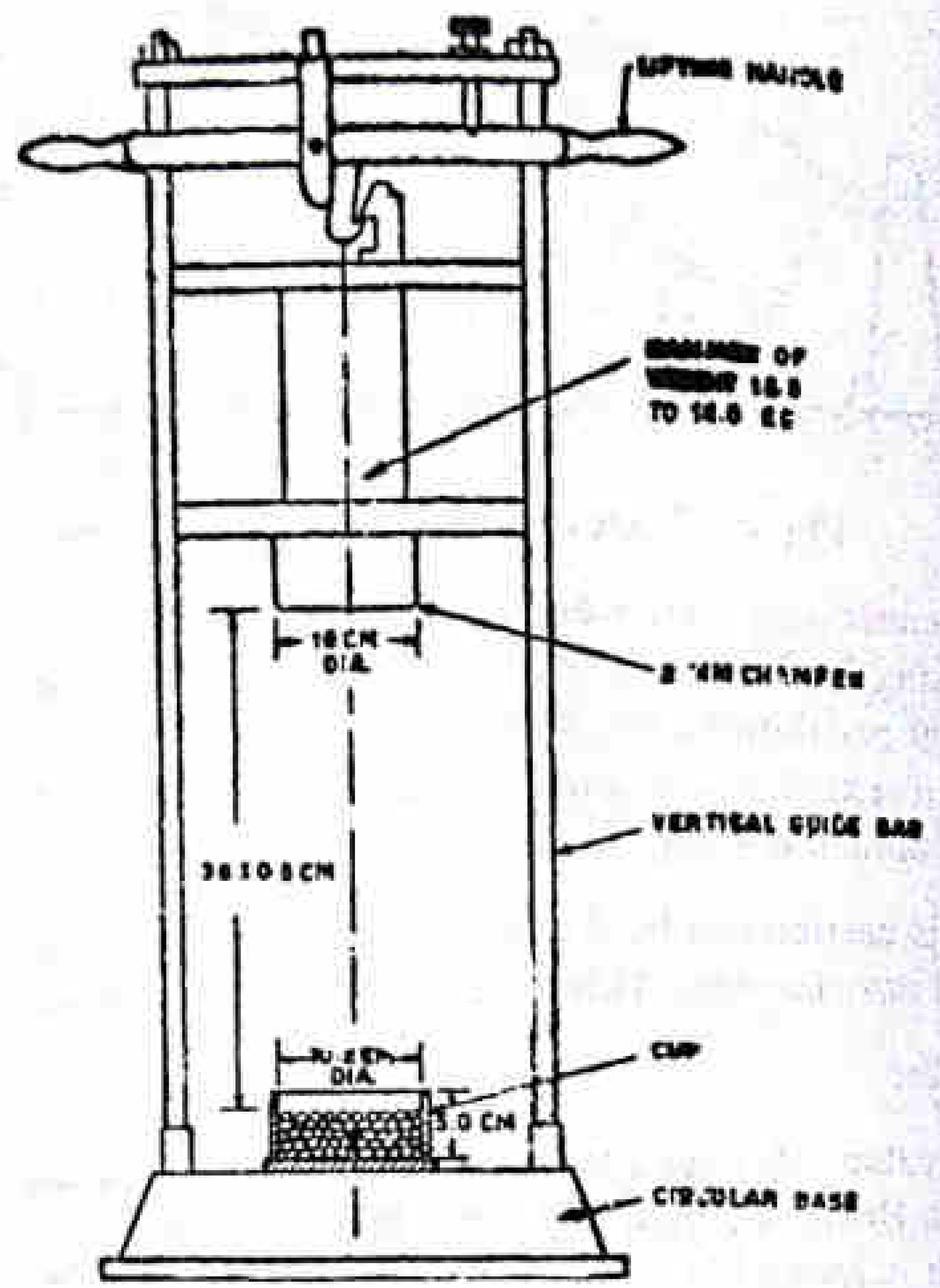


Fig. 6.18 Aggregate Impact Testing Machine

Soundness test

Soundness test is intended to study the resistance of aggregates to weathering action, by conducting accelerated weathering test cycle. In order, to quicken the effects of weathering due to alternate wet-dry and/or freeze-thaw cycles in the laboratory, the resistance to disintegration of aggregate is determined by using saturated solution of *sodium sulphate* or *magnesium sulphate*. Clean, dry aggregate specimen of specified size range is weighed and counted. It is immersed in the saturated solution of sodium sulphate or magnesium sulphate for 16 to 18 hours. Then the specimen is dried in an oven at 105-110°C to a constant weight thus making one cycle of immersion and drying. The number of such cycles is decided by prior agreement and then the specimens are tested. After completing the final cycle, the sample is dried and each fraction of the aggregate is examined visually to see if there is any evidence of excessive splitting, crumbling or disintegration of the grains. Sieve analysis is carried out to note the variation in gradation from the original. The coarse aggregate fraction of each size range is sieved on specified sieve sizes. The average loss in weight of aggregates to be used in pavement construction after 10 cycles should not exceed 12 percent when tested with sodium sulphate and 18 percent when tested with magnesium sulphate.

Shape tests

The particle shape of aggregate mass is determine by the percentages of flaky and elongated particles contained in it and by its angularity. The evaluation of shape of the particles made in terms of flakiness index, elongation index and angularity number.

Flakiness Index

The flakiness index of aggregate is the percentage by weight of aggregate particles whose least dimension/thickness is less than three fifths or 0.6 of their mean dimension.

The test is applicable to sizes larger than 6.3 mm. Standard thickness gauge is used to gauge the thickness of the samples. The sample of aggregates to be tested is sieved through a set of sieves and separated into specified size ranges. Now to separate the flaky material, the aggregates which pass through the appropriate elongated slot of the thickness gauge are found. The width of the appropriate slot would be 0.6 of the average of the size range. If the size range of aggregate in a group is 16-20 mm, the width of the slot to be selected in thickness gauge would be $18 \times 0.6 = 10.8$ mm. The flaky material passing the appropriate slot from each size range of test aggregates are added up and let this weight be w. If the total weight of sample taken from the different size ranges is W, the flakiness index is given by 100 w/W percent, or in other words it is the percentage of flaky materials, the widths of which are less than 0.6 of the mean dimensions. It is desirable that the flakiness index of aggregates used in road construction is less than the 15 percent and normally does not exceed 25 percent.

Elongation Index

The elongation index of an aggregate is the percentage by weight of particles whose greatest dimension or length is greater than one and four fifth or 1.8 times their mean dimension. The elongation test is not applicable for sizes smaller than 6.3 mm.

The sample of aggregate to be tested is sieved through a set f sieve and separated into specified size ranges. The aggregates from each of the size range is then individually passed through the appropriate gauge of the length gauge with the longest side in order to separate the elongated particles. The gauge length would be 1.8 times the mean size of the aggregate. The portion of the elongated aggregate having length greater than the specified gauge from each range is weighed and the total weight of the elongated stones, is expressed as a percentage of the total weight of the sample, to get the elongation index.

Elongated and flaky aggregates are less workable; they are also likely to break under smaller loads than the aggregate which are spherical or cubical. Flakiness index and elongation index values in excess of 15 percent are generally considered undesirable; however no recognised limits have been laid down for elongation index.

Angularity Number

Based on the shape of the aggregate particles, they may be classified as rounded, irregular or partly rounded, angular and flaky. Angular particles possess well defined edges formed at the intersection of roughly plane faces and are commonly found in aggregates prepared by crushing of rocks. Since weaker aggregates may be crushed during compaction, the angularity number does not apply to any aggregate which breaks down during this test. Angularity or absence of rounding of the particles of an aggregate is a property which is of importance because it affects the ease of handling a mixture of aggregate and binder. The determination of angularity number of an aggregate is essentially a laboratory method intended for comparing the properties of different aggregates for mix design purposes.

The degree of packing of particles of single sized aggregates depends on the shape and angularity of the aggregate. Hence the angularity of the aggregate can be estimated from the properties of voids in a sample of aggregate compacted in a particular manner. Angularity number is defined as 67 - percent solid volume. The solid volume of the aggregate is found by filling it in a vessel in a specified manner. In the expression for angularity number, the value 67 represents the volume of solids (in percent) of most rounded gravel in a well compacted state which would then have 33 percent voids. Thus the angularity number measures the voids in excess of 33 percent. The higher the number, more angular is the aggregate. The range of angularity number for aggregates used in constructions is 0 to 11.

The apparatus for testing the angularity number consists of a metal cylinder of capacity 3 litre, tamping rod and a metal scoop. The test sample is sieved and a specified size ranges of the aggregate, such as 16 – 20 mm, 12.5 – 16 mm, etc. are used for the test. A scoop full of this single size aggregate is placed in the cylinder and tamped 100 times by the rod. Second and third layers are placed and tamped similarly and the excess aggregate is struck off level to the top surface of the cylinder. The weight of aggregate in the cylinder is found to be Wg. Then the cylinder is found = Cg. The specific gravity Ga of the aggregate is also determined. The angularity number is found from the formula:

$$\text{Angularity number} = 67 - \frac{100\,W}{CGa} \qquad (6.9)$$

This value is expressed as the nearest whole number.

Specific gravity and water absorption tests

The specific gravity of an aggregate is considered to a measure of the quality or strength of the material. Stones having low specific gravity values are generally weaker than those having higher values. The specific gravity test also helps identifying the stone specimen. Stones having higher water absorption value are porous and thus weak. They are generally unsuitable unless found acceptable based on crushing and hardness tests.

About 2 kg of dry aggregate sample is placed in wire basket and immersed in water for 24 hours. The sample is weighed in water and the buoyant weight is found. The aggregates are then taken out weighed after drying the surface. Then the aggregates are dried in an oven for 24 hours at a temperature 100 - 110°C, and then the dry weight is determined. The specific gravity is calculated by dividing the dry weight of aggregate by weight of equal volume of water. The water absorption is expressed as the percent water absorbed in terms of over dried weight of the aggregates.

The specific gravity of rocks very from 2.6 to 2.9. Rock specimens having more than 0.6 percent water absorption are considered unsatisfactory unless found acceptable based on strength tests. However slightly higher value of porosity may be acceptable for aggregates used in bituminous pavement construction, if the aggregates are found otherwise suitable.

Bitumen adhesion test

Bitumen and tar adhere well to all normal types of road aggregates provided they are dry and are free from dust. The process of initial binding is controlled largely by the viscosity of the binder. In the absence of water there is practically no adhesion problem in bituminous construction. The problems are observed due to the presence of water. First if aggregate is wet and cold, it is normally not possible to coat with a bituminous binder. This problem can be dealt-with by removing the water film on the aggregate by drying, and by increasing the mixing temperature. Second problem is *stripping* of binder from coated aggregate due to presence of water. This problem of *stripping* is generally experienced only with bituminous mixtures which are permeable to water. The stripping is due to the fact that some aggregates have greater affinity towards water than with bituminous binders and this displacement depends on the physico-chemical forces acting on the system.

Most road stones have surfaces that are electrically charged. As an example silica a common constituent of igneous rocks possess a weak negative charge and hence these have greater attraction with the polar liquid water than with bituminous binders having little polar activity. These aggregates which are electronegative are water-linking and are called *hydrophillic*. Basic aggregates like lime-stones have a dislike for water and greater attraction to bitumen, as they have positive surface charge. These aggregates are called *hydrophobic*.

It is important to know the type of charge of aggregates used in bituminous construction. Now bitumen is also available as cationic or positive and anionic or negative and hence a suitable selection may be made depending on aggregates available. Cationic (+) bitumen may be selected for electronegative aggregate and anionic (–) bitumen for electropositive aggregates.

Several laboratory tests have been developed to arbitrarily determine the adhesion of bituminous binder to an aggregate in presence of water. These tests may be classified into six types.

(i) Static immersion test

(ii) Dynamic immersion test

(iii) Chemical immersion test

(iv) Immersion mechanical test

(v) Immersion trafficking test and

(vi) Coating test

The static immersion test is very commonly used as it is quite easy and simple. The principle of this type of test is by immersing aggregate fully coated with the binder in water maintained at specified temperature and by estimating the degree of stripping. The result is reported as the percentage of stone surface that is stripped off after the specified time periods. IRC has specified that stripping value of aggregates should not exceed 25 percent for use in bituminous surface dressing, penetration macadam, bituminous macadam and carpet constructions, when aggregate coated with bitumen is immersed in water bath at 40°C for 24 hours.

6.3 BITUMINOUS MATERIALS

6.3.1 Introduction

Bituminous binders used in pavement construction works include both bitumen and tar. Bitumen is a petroleum product obtained by the distillation of petroleum crude where-as road tar is obtained by the destructive distillation of coal or wood. Both bitumen and tar have similar appearance, black in colour though they have different characteristics. Both these materials can be used for pavement works.

Bitumen is hydrocarbon material of either natural or pyrogenous origin, found in gaseous, liquid, semisolid or solid form and is completely soluble in *Carbon disulphide* and in *carbon tetra chloride*. Bitumen is a complex organic material and occurs either naturally or may be obtained artificially during the distillation of petroleum. Bituminous materials are very commonly used in highway construction because of their binding and their water proofing properties.

When the bitumen contains some inert material or *minerals*, it is some times called asphalt. Asphalt is found as deposits in the form of natural asphalt or rock asphalt.

The grades of bitumen used for pavement construction work of roads and airfields are called paving grades and those used for water proofing of structures and industrial floors etc. are called industrial grades. The paving bitumen available in India are classified into two categories :

(i) paving bitumen from Assam petroleum, denoted as A-type and designated as grades A35, A 90, etc.

(ii) paving bitumen from other sources denoted as S-type and designated as grades S 35, S 90, etc.

6.3.2 Types of Bituminous Materials

Bituminous material used in highway construction may be broadly divided as :

(i) Bitumen and

(ii) Tar

Bitumen may be further divided as petroleum asphalt or bitumen and native asphalt.

There are different forms in which native asphalts are available. Native asphalts are those which occur in a pure or nearly pure state in nature. Native asphalts which are associated with a large proportion of mineral matter are called rock asphalts. The viscosity of bitumen is reduced some times by a volatile diluent; this material is called *cutback*. When bitumen is suspended in a finely divided condition in an aqueous medium and stabilized with an emulsifier, the material is known as *emulsion*. Tar is the viscous liquid obtained when natural organic materials such as wood and coal are carbonized or destructively distilled in the absence of air. Processing of bitumen and bituminous products is diagrammatically represented in Fig. 6.19.

6.3.3 Bitumen

Crude petroleum obtained from different places are quite different in their composition. The portion of bituminous material present in the petroleums may widely differ depending on the source. Almost all the crude petroleums contain considerable amounts of water along with crude oil. Hence the petroleum should be dehydrated first before carrying out the distillation. General types of distillation processes are fractional distillation and destructive distillation. In fractional distillation the various volatile constituents are separated at successively higher temperatures without substantial chemical change. The successive fractions obtained yield gasoline, naptha, kerosene and lubricating oil; the residue would be petroleum bitumen. In destructive distillation the material undergoes chemical change under the application of extreme heat and pressure. The process is usually applied for the manufacture of tar. Steam distillation of petroleum is employed to produce steam refine petroleum bitumen in order to remove high boiling point constituents such as heavy lubricating oils without causing chemical changes. When the residue is distilled to a definite consistency without further treatment, the bitumen obtained as residue is called *straightrun bitumen*.

Requirements of Bitumen

The desirable properties of bitumen depend on the mix type and the construction. The general problems while using bitumen in paving mixes are :

(i) mixing

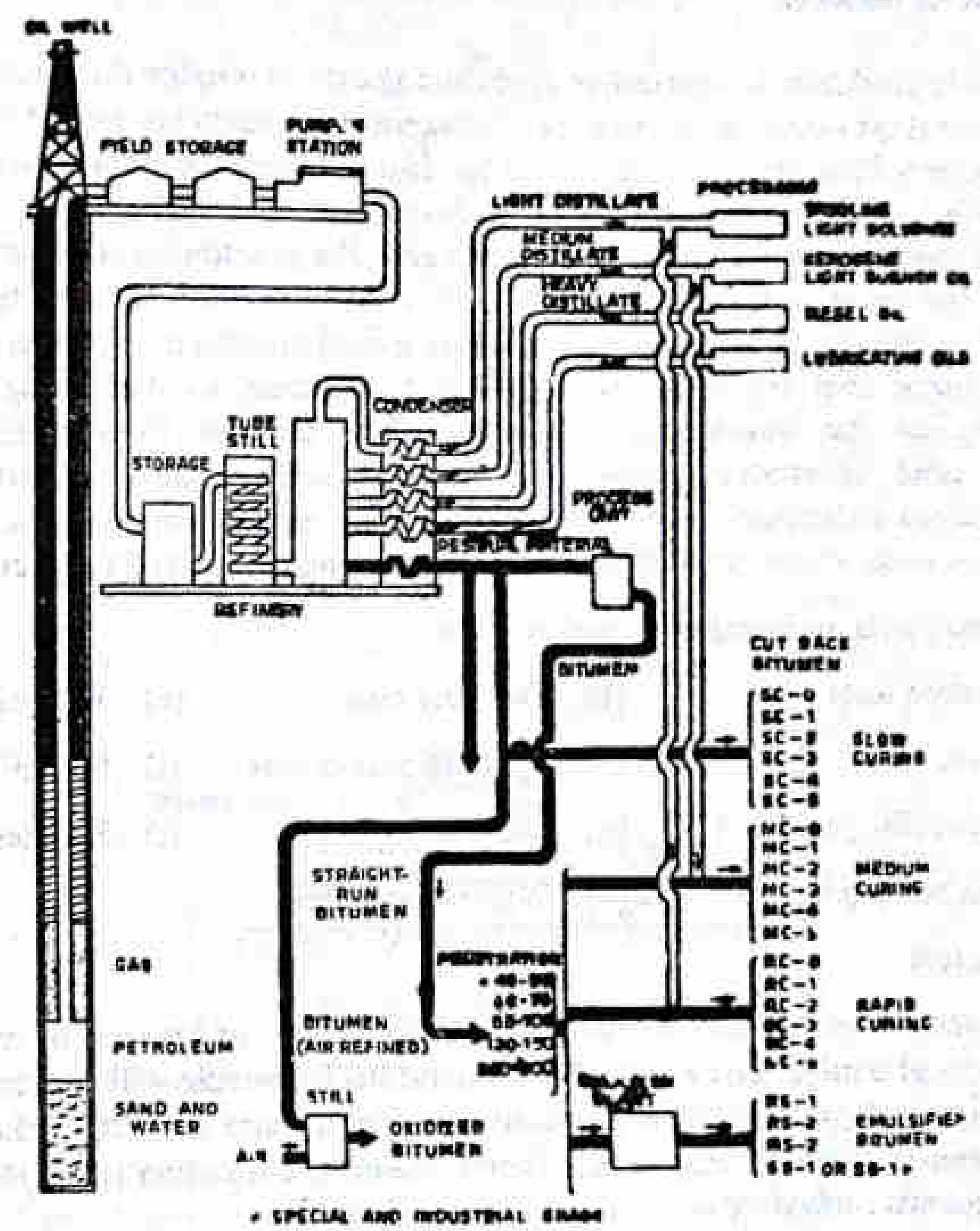


Fig. 6.19 Processing of Bituminous Products

(ii) attainment of desired stability of the mix

(iii) to maintain the stability under adverse weather conditions

(iv) to maintain sufficient flexibility and thus avoid cracking of bituminous surface and

(v) to have sufficient adhesion with the aggregates in the mix in presence of water.

In view of the above problems, the bitumen should possess the following desirable properties :

(i) The viscosity of the bitumen at the time of mixing and compaction should be adequate. This is achieved by heating the bitumen and aggregate prior to mixing or by use of cutbacks or emulsions of suitable grade.

(ii) The bituminous material should not be highly temperature susceptible. During the hottest weather of the region the bituminous mix should not become too soft or unstable. During cold weather the mix should not become too hard and brittle, causing cracking of surface. The material should be durable.

(iii) In presence of water the bitumen should not strip off from the aggregate. There has to be adequate affinity and adhesion between the bitumen and aggregate used in the mix.

6.3.4 Tests on Bitumen

Bitumen is available in a variety of types and grades. To judge the suitability of these binders various physical tests have been specified by agencies like ASTM, Asphalt Institute, British Standards Institution and the ISI. These tests include penetration tests, ductility tests, softening point test and viscosity test. For classifying bitumen and studying the performance of bituminous pavements, the penetration and ductility tests are essential. The other tests like softening point and flash and fire point tests are more important to guide the paving technologists during field operations. In recent years, it has been recognized that the above tests are not sufficient to define the temperature susceptibility of the bituminous materials. The bitumen from different sources possessing same penetration value at a specified temperature may exhibit entirely different viscous characteristics at the application or service temperatures. These tests therefore may need intensive correlation with fundamental property like viscosity.

The various tests on bituminous materials are :

(a) Penetration tests
(b) Ductility tests
(c) Viscosity tests
(d) Float test
(e) Specific gravity test
(f) Softening point test
(g) Flash and Fire point test
(h) Solubility test
(i) Spot test
(j) Loss on heating test
(k) Water content test

Penetration test

The penetration test determines the hardness or softness of bitumen by measuring the depth in tenths of a millimetre to which a standard loaded needle will penetrate vertically in five seconds. The sample is maintained at a temperature of 25°C. The concept of penetration test is shown in Fig. 6.20. Indian Standard Institution has standardized the equipment and test procedure.

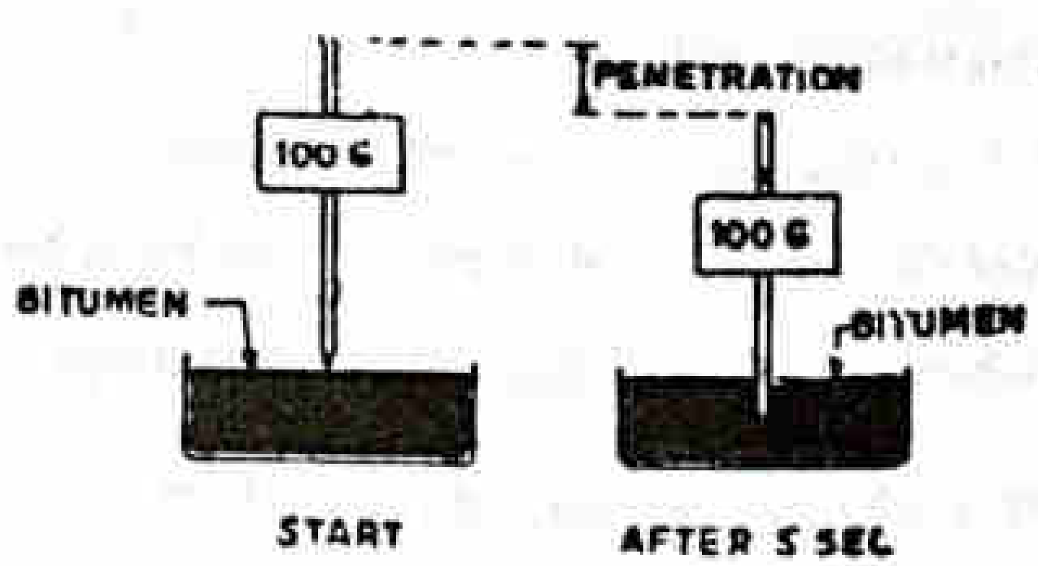


Fig. 6.20 Penetration Test Concept

The penetrometer consists of a needle assembly with a total weight of 100 g and device for releasing and locking in any position. There is a graduated dial to read penetration values to 1/10th of a millimeter. Refer Figure 6.21.

The bitumen is softened to a pouring consistency, stirred thoroughly and poured into containers to a depth at least 15 mm in excess of the expected penetration. The sample containers are then placed in a temperature controlled water bath at a temperature of 25°C for one hour. The sample with container is taken out and the needle is arranged to make contact with the surface of the sample. The dial is set to zero or the initial reading is

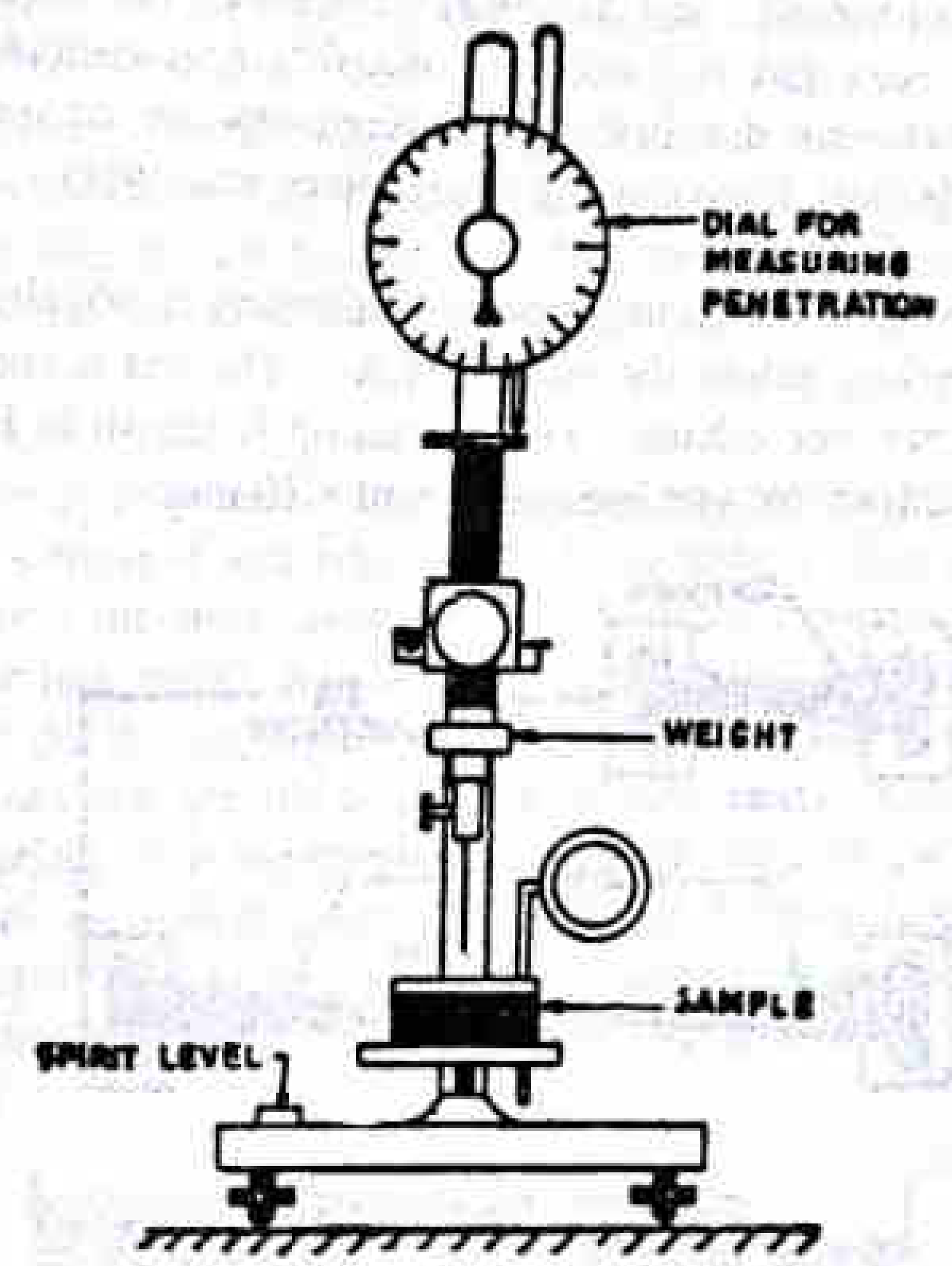


Fig. 6.21 Penetrometer

taken and the needle is released for 5 seconds. The final reading is taken on dial gauge. At least three penetration tests are made on this sample by testing at distances of atleast 10 mm apart. After each test the needle is disengaged and wiped with benzene and dried. The depth of penetration is reported in one-tenth millimeter units. The mean value of three measurements is reported as a penetration value. It may be noted that the penetration value is largely influenced by any inaccuracy as regards pouring temperature, size of needle, weight placed on the needle and the test temperature.

The bitumen grade is specified in terms of penetration value. 80-100 or 80/100 grade bitumen means that the penetration value of the bitumen is in the range 80 to 100 at standard test conditions. The penetration test is applied almost exclusively to bitumen. As road tars are soft, the penetration test cannot be carried out on these materials. Other consistency tests are used for tars, cutbacks and emulsions.

The penetration values of various types of bitumen used in pavement construction in this country range between 20 and 225. 30/40 and 80/100 grade bitumen are more commonly used, depending on construction type and climatic conditions. In hot climates a lower penetration grade bitumen like 30/40 bitumen is preferred.

Ductility test

In the flexible pavement constructions where bitumen binders are used, it is important that the binders form ductile thin films around the aggregates. This serves as a satisfactory binder in improving the physical interlocking of the aggregate bitumen mixes. Under traffic loads the bituminous pavement layer is subjected to repeated deformation and recoveries. The binder material which does not possess sufficient ductility would crack and thus provide pervious pavement surface. Ductility test is carried out on bitumen to test this property of the binder. The test is believed to measure the adhesive property of bitumen and its ability to stretch. The bitumen may satisfy the penetration value, but may fail to satisfy the ductility requirements. Bitumen paving engineer would however want that both test requirements are satisfied in the field jobs. Penetration and ductility tests cannot in any case replace each other.

The ductility is expressed as the distance in centimeters to which a standard briquette of bitumen can be stretched before the thread breaks. The test is conducted at 27°C and at a rate of pull of 50 mm per minute. The test set up is shown in Fig. 6.22. The cross section at minimum width of the specimen is 10 mm × 10 mm.

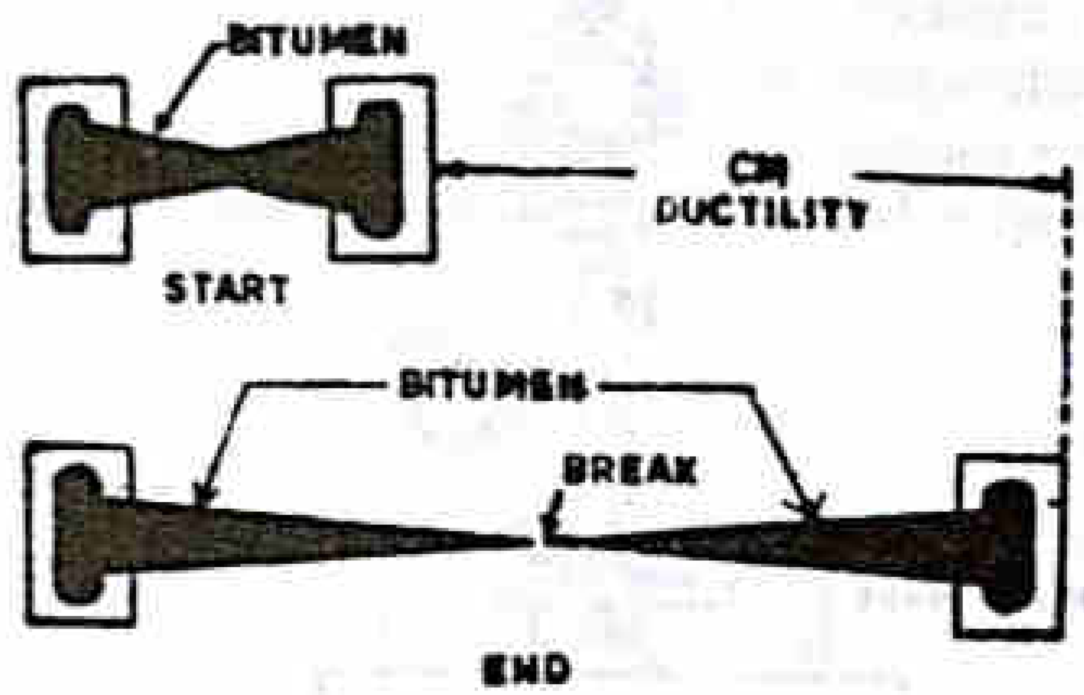


Fig. 6.22 Ductility Test

The ductility machine functions as a constant temperature water bath with a pulling device at a pre-calibrated rate. Two clips are thus pulled apart horizontally at a uniform speed of 50 mm per minute.

The bitumen sample is heated and poured in the mould assembly placed on a plate. The samples along with the moulds are cooled in air and then in water bath maintained at 27°C. The excess bitumen material is cut and the surface is leveled using a hot knife. The mould assembly containing sample is replaced in water bath of the ductility testing machine for 85 to 95 minute. The sides of the mould are removed, the clips hooked on the machine and the pointer is adjusted to zero. The distance upto the point of breaking of thread is reported in centimeters as ductility value. The ductility value gets seriously affected by factors such as pouring temperature, dimensions of briquette, level of briquette in the water bath, presence of air pockets in the modulus briquettes, test temperature and rate of pulling.

The ductility values of bitumen vary from 5 to over 100 for different bitumen grades. A minimum ductility value of 75 cm has been specified by the ISI for bitumens of grades 45 and above, obtained from sources other than Assam Petroleum (i.e., S 45, and above), the minimum ductility value may be 50 cm for bitumens of grades S 35, obtained from these sources. However, as the bitumen produced from Assam Petroleum in India have much lower ductility values, the minimum ductility value specified is only 15 cm for the bitumen grades A 65 to 200 for use in certain regions.

Viscosity test

Viscosity is defined as inverse of fluidity. Viscosity thus defines the fluid property of bituminous material. Viscosity is the general term for consistency and it is measure of resistance to flow. Many researchers believe that grading of bitumen should be by absolute viscosity units instead of the conventional penetration units.

The degree of fluidity of the binder at the application temperature greatly influences the strength characteristics of the resulting paving mixes. High or low viscosity during mixing or compaction has been observed to result in lower stability values. There is an optimum value of viscosity for each aggregate gradation of the mix and bitumen grade. At low viscosity, the bituminous binder simply *lubricates* the aggregate particles instead of providing a uniform film for binding action. Similarly high viscosity also resists the compactive effort and the resulting mix is heterogeneous in character exhibiting low stability values.

Orifice type viscometer may be used to indirectly find the viscosity of liquid binders like cutback bitumen, emulsion and liquid tar. According to this method, viscosity is measured by determining the time taken by 50 ml of the material to flow from a cup through a specified orifice under standard test conditions and specified temperature. Higher the viscosity of the binder, higher will be the time required. This is illustrated in Fig. 6.23. Furol viscosity is a specific test which is used only to measure the viscosity of liquid bituminous materials. It is the number of seconds required for 50 ml of material to flow through an orifice of specified size at specified temperatures. Equipment like sliding plate microviscometer, and Brookfield viscometer are however in use for defining the viscous characteristics of the bitumen of all grades irrespective of testing temperature.

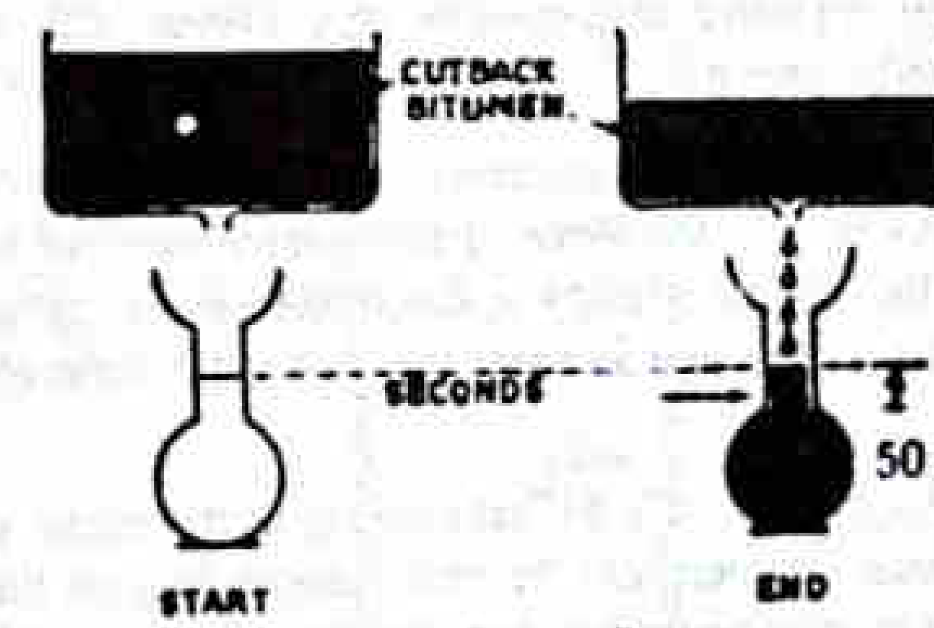


Fig. 6.23 Viscosity Test set-up

The viscosity of tar is determined as the time taken in seconds for 50 ml of the sample to flow through 10 mm orifice of the standard tar viscometer at the specified temperature of 35, 40, 45 or 55°C. The viscosity of cutback bitumen is determined as the time taken in seconds for 50 ml of the sample to flow through either 4.0 mm orifice at 25°C or 10 mm orifice at 25 or 40°C. Thus the orifice viscometer is suitable to test tars and cutbacks.

Float test

There is a range consistency of the bituminous materials for which neither an orifice viscometer test nor a penetration test could be used to define the consistency of the material. The consistency of materials of this group is measured by float test.

The apparatus consists of a float made of aluminum and a brass collar filled with the specimen materials to be tested, which is screwed to the float. The test specimen is filled in the collar (mould), cooled to a temperature of 5°C and screwed into the float. Refer Fig. 6.24. The float assembly is floated in a water bath at 50°C and the time required in seconds for water to force its way through the bitumen plug is noted as the float test value. The higher the float test value, the stiffer is the material.

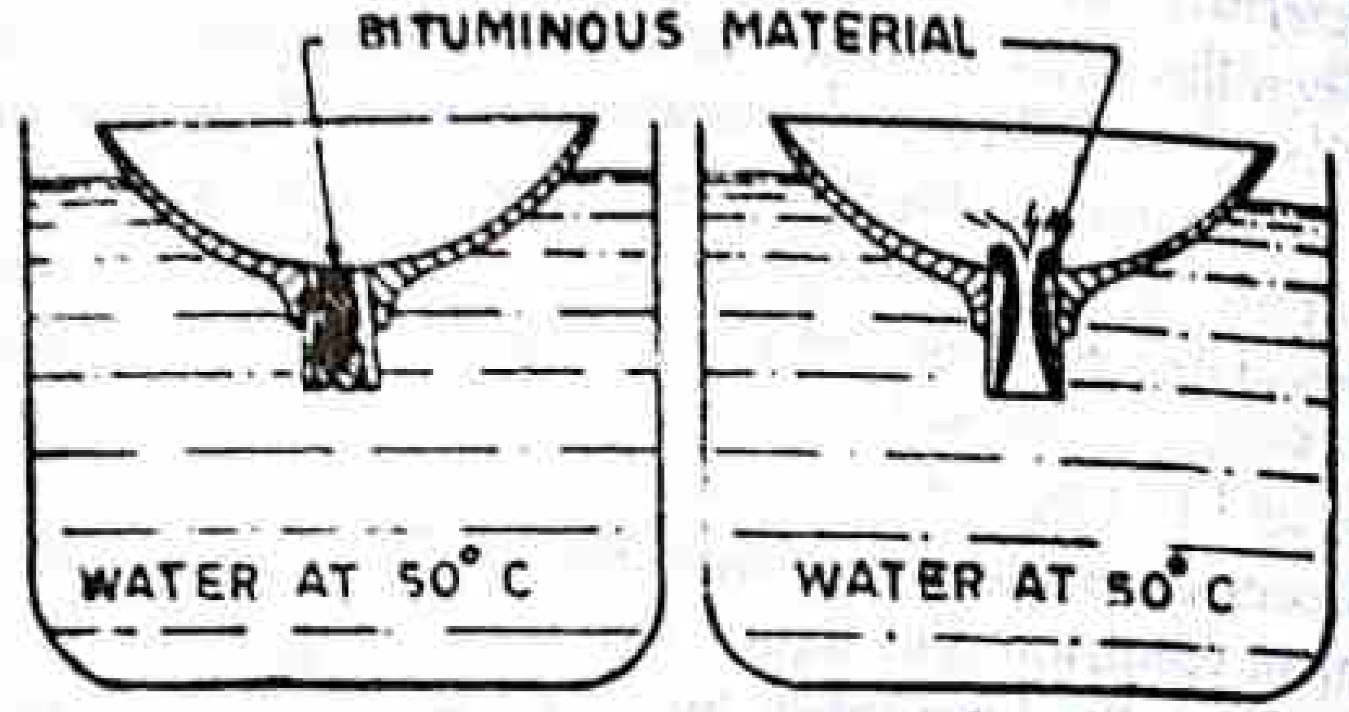


Fig. 6.24 Float Test Set-up

Specific gravity test

The density of a bitumen binder is a fundamental property frequently used as an aid to classify the binders for use in paving jobs. In most applications, the bitumen is weighed, but finally when used with aggregate system, the bitumen content is converted on volume basis using density values. The specific gravity value of bitumen is also useful in bituminous mix design. The density of bitumen is greatly influenced by its chemical composition. Increased amounts of *aromatic type* compounds or mineral impurities cause an increase in specific gravity.

The specific gravity of bituminous materials is defined as the ratio of the mass of a given volume of the substance to the same of an equal volume of water, the temperature of both being 27°C. The specific gravity is determined either by using a pycnometer or by preparing a cube shape specimen in semi solid or solid state and by weighing in air and water.

Generally the specific gravity of pure bitumen is in the range of 0.97 to 1.02. The specific gravity of cutback bitumen may be lower depending on the type and proportion of diluent used. Tars have specific gravity ranging from 1.10 to 1.25.

Softening point test

The softening point is the temperature at which the substance attains a particular degree of softening under specified condition of test. The softening point of bitumen is usually determined by Ring and Ball test. The test set-up is shown in Fig. 6.25.

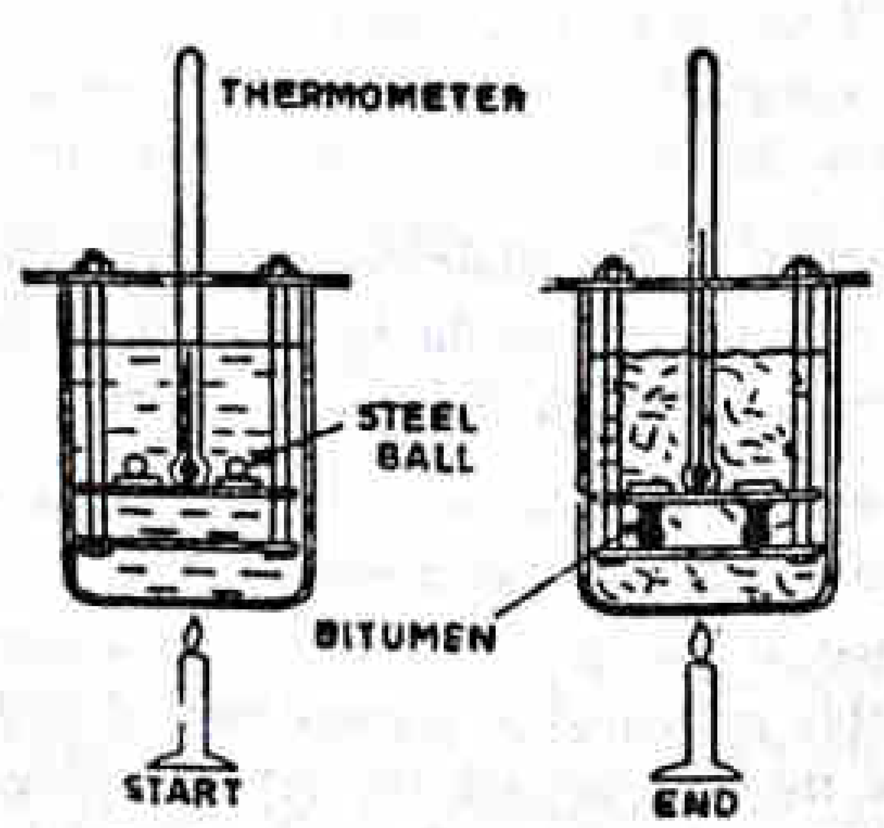


Fig. 6.25 Softening Point Test Set-up

Generally higher softening point indicates lower temperature susceptibility and is preferred in warm climates. A brass ring containing test sample of bitumen is suspended in liquid like water or glycerine at a given temperature. A steel ball is placed upon the bitumen sample and the liquid medium is then heated at a rate of 5°C per minute. The temperature at which the softened bitumen touches the metal placed at a specified distance below the ring is recorded as the softening point of a bitumen. Hard grade bitumen possess higher softening point than soft grade bitumens.

The softening point of various bitumen grades used in paving jobs vary between 35° to 70°C.

Flash and fire point test

Bitumen materials leave out volatiles at temperatures depending upon their grade. These volatiles catch fire causing a flash. This condition is very hazardous and it is therefore essential to qualify this temperature for each bitumen grade, so that paving engineers may restrict the mixing and application temperatures. As mentioned above, this test gives an indication of the critical temperature at and above which suitable precaution should be taken to eliminate fire hazards during heating of bitumen. The definition of flash and fire points as given by the ISI are as follows :

Flash point "The flash point of a material is the lowest temperature at which the vapour of a substance momentarily takes fire in the form of a flash under specified condition of test".

Fire point "The fire point is the lowest temperature at which the material gets ignited and burns under specified conditions of test". Refer Fig. 6.26 for the test set up.

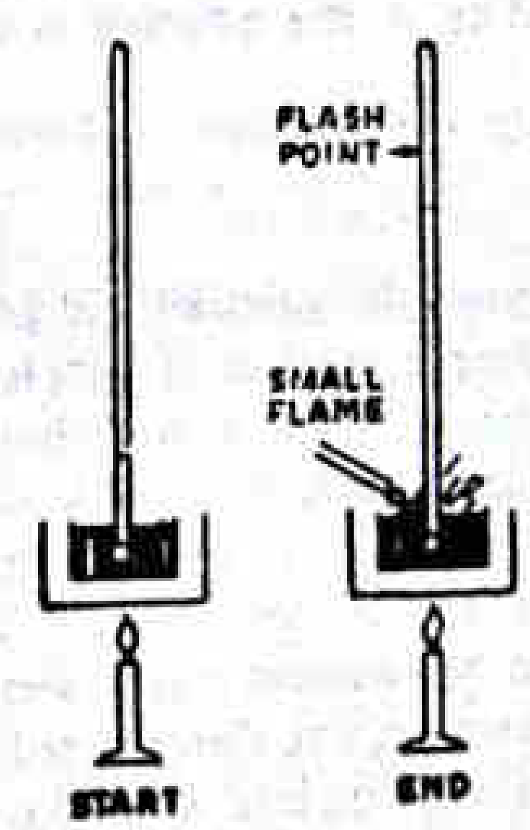


Fig. 6.26 Flash Point Test Set-up

Pensky-Martens closed cup apparatus or open cup are used for conducting the tests. The material to be tested is filled in the cup upto a filling mark. The lid is placed to close the cup in a closed system. All accessories including thermometer of the specified range are suitably fixed. The bitumen sample is then heated at the rate of 5° to 6°C per minute, stirring the specimen. The test flame is applied at intervals depending upon the expected flash and fire points. First application is made at least 17°C below the actual flash point and then at every 1° to 3°C.

The flash point is taken as the temperature read on the thermometer at the time of the flame application that causes a bright flash in the interior of the cup in closed system.

For open cup it is the instance when flash appear first at any point on the surface of the material. The heating is continued until the material gets ignited and continues to burn for 5 seconds; this temperature is recorded as the fire point.

The minimum specified flash point of bitumen used in pavement construction in Pensky Martens closed type test is 175°C.

Solubility test

Pure bitumen is completely soluble in solvents like carbon *disulphide* and *carbon tetrachloride*. Hence any impurity in bitumen in the form of inert minerals, carbon, salts etc. could be quantitatively analysed by dissolving the samples of bitumen in any of the two solvents. A sample of about 2 g of bitumen is dissolved in about 100 ml of solvent. The solution is filtered and the insoluble material retained is washed, dried and weighed; it is expressed as a percentage of original sample. The insoluble material should be preferably less than 1.0 percent. In solubility test with carbon tetrachloride, if black carbonaceous residue is over 0.5 percent, the bitumen is considered to be *cracked*. The minimum proportion of bitumen soluble in carbon disulphide is specified as 99 percent.

Spot test

This is a test for detecting over heated or *cracked* bitumen. This test is considered to be more sensitive than the solubility test for detection of cracking. About 2 g of bitumen is dissolved in 10 ml of *naphtha*. A drop of this solution is taken out and placed on a filter paper, one after one hour and second after 24 hours after the solution is prepared. If the strain of the spot on the paper is uniform in colour, the bitumen is accepted as uncracked. But if the spots form dark brown or black circle in the centre with an annular ring of lighter colour surrounding it, the bitumen is considered to be over heated or cracked.

Loss on heating test

When bitumen is heated, it loses the volatiles and gets hardened. To study the effect of heating, an accelerated heating procedure is adopted. About 50g of the sample is weighed and heated to a temperature of 163°C for 5 hours in a special oven designed for this test. This specimen is weighed again after the heating period and the loss in weight is expressed as a percentage by weight of original sample. Bitumen used in pavement mixes should not indicate more than one percent loss in weight; for bitumen of penetration values 150-200 upto two percent loss in weight is allowed. The residue after heating when subjected to penetration test shows a reduction in penetration value. The reduction in penetration value should be less than 40 percent of the original penetration value of the bitumen.

Water content test

It is desirable that the bitumen contains minimum water content to prevent foaming of the bitumen when it is heated above the boiling point of water. The water content in a bitumen is determined by mixing known weight of the specimen in a pure petroleum distillate free from water, heating and distilling off the water. The weight of the water condensed and collected is expressed as percentage by weight of the original sample. The maximum water content in bitumen should not exceed 0.2 percent by weight.

Explanations for some of the above tests have been based on the publication of *Burmah Shell* (See list of references).

6.3.5 Cutback Bitumen

Cutback bitumen is defined as the bitumen, the viscosity of which has been reduced by a volatile diluent. For use in surface dressings, some type of bitumen macadam and soilbitumen stabilization, it is necessary to have a fluid binder which can be mixed relatively at low temperatures. Hence to increase fluidity of the bituminous binder at low temperatures the binder is blended with a volatile solvent. After the cutback mix is used in construction work, the volatile gets evaporated and the cutback develops the binding properties. The viscosity of the cutback and rate of which it hardens on the road depend on the characteristics and quantity of both bitumen and volatile oil used as the diluent. Cutback bitumens are available in three types, namely,

(i) Rapid Curing (RC)

(ii) Medium Curing (MC) and

(iii) Slow Curing (SC)

This classification is based on the rate of curing or hardening after the application. The grade of cutback or its fluidity is designed by a figure which follows the initials; as an example RC-2 means that it is a rapid curing cutback of grade 2.

The cutback with the lowest viscosity is designated by numeral 0, such as RC-0, MC-0 and SC-0. Suffix numerals 0, 1, 2, 3, 4 and 5 designate progressively thicker or more viscous cutbacks as the numbers increase. This number indicates a definite viscosity irrespective of the type of cutback; in other words, RC-2, MC-2 and SC-2 all have the same initial viscosity at a specified temperature. The initial viscosity values (in seconds, standard tar viscometer) of various grades of cutbacks as per ISI specifications are given in Table 6.7.

Table 6.7 Viscosity of Cutbacks

Type and grade of Cutback	Viscosity in seconds in tar viscometer		
	4 mm orifice 25°C	10 mm 25°C	10 mm 40°C
RC-0, MC-O & SC-0	25 to 75		
RC-1, MC-1 & SC-1	50 to 100		
RC-2, MC-2 & SC-2		10 to 20	
RC-3, MC-3 & SC-3		25 to 75	
RC-4, MC-4 & SC-4			14 to 45
RC-5, MC-5 & SC-5			60 to 100

Thus lower grade cutbacks like RC-0, RC-1 etc. would contain high proportion of solvent when compared with higher grades like RC-4 or RC-5. RC-0 and MC-0 may contain approximately 45 percent solvent and 55 percent bitumen, whereas, RC-5 and MC-5 may contain approximately15 percent solvent and 85 percent bitumen.

Rapid Curing Cutbacks are bitumens, fluxed or cutback with a petroleum distillate such as nephta or gasoline which will rapidly evaporate after using in construction, leaving the bitumen binder. The grade of the R.C. cutback is governed by the proportion of the solvent used. The penetration value of residue from distillation up to 360°C of RC cutback bitumen is 80 to 120.

Medium curing cutbacks are bitumen fluxed to greater fluidity by blending with a intermediate-boiling-point solvent like kerosene or light diesel oil. MC cutbacks evaporate relatively at slow rate because the kerosene-range solvents will not evaporate rapidly as the gasoline-range solvents used in the manufacture of RC cutbacks. Hence the designation 'medium curing' is given to this cutback type. MC products have good wetting properties and so satisfactory coating of fine grain aggregate and sandy soils is possible.

Slow curing cutbacks are obtained either by blending bitumen with high-boiling-point gas oil, or by controlling the rate of flow and temperature of the crude during the first cycle of refining. SC cutbacks or wood soils harden or set way slowly as it is a semi volatile material.

Various tests carried out on cut-backs bitumen are :

(a) Viscosity tests at specified temperature using specified size of orifice.

(b) Distillation test to find distillation fractions, up to specified temperature and to find the residue from distillation up to 360°C

(c) Penetration test, ductility test and test for matter soluble in carbon disulphide on residue from distillation up to 360°C

(d) Flash point test on cutback using Pensky Martens closed type apparatus.

6.3.6 Bituminous Emulsion

A bitumen emulsion is liquid product in which a substantial amount of bitumen is suspended in a finely divided condition in an aqueous medium and stabilized by means of one or more suitable materials. An emulsion is a two phase system consisting of two immiscible liquids; the one being dispersed as fine globules in the other.

Usually, bitumen or refined tar is broken up into fine globules and kept in suspension in water. A small proportion of an emulsifier is used to facilitate the formation of dispersion and to keep the globules of dispersed binder in suspension. The function of this emulsifier is to form a protective coating around the globules of binder resisting the coalescence of the globules. Emulsifiers usually adopted are soaps, surface active agents and colloidal powders. Half to one percent emulsifier by weight of finished emulsion are usually taken while preparing normal road emulsions. The bitumen/tar content of emulsions range from 40 to 60 percent and the remaining portion is water. The average diameter of globules of bitumen portion is about two microns.

Usually the bitumen grades which are emulsified for road construction works are those with penetration values between 190 and 320. Emulsions of tar and tar bitumen mixture are also prepared, but their use is restricted. Two methods commonly followed for the preparation of emulsions are the colloidal mill method and the high-speed mixer method. The manufactured emulsions are stored in air tight drums.

When the emulsion is applied on the road, it breaks down and the binder starts binding the aggregates, though the full binding power develops slowly as and when the water evaporates. The first sign of break down of emulsion is shown by the change in colour of the film from chocolate brown to black. If the bitumen emulsion is intended to break rapidly, the emulsion is said to possess rapid-set quality. Emulsions which do not break spontaneously on contact with stone, but break during mixing or by fine mineral dust are medium-set grades. When special types of emulsifying agents are used to make the emulsion relatively stable, they are called slow setting grades.

Emulsions are used in bituminous road constructions, especially in maintenance and patch repair works. The main advantage of emulsion is that it can be used in wet weather even when it is raining. Also emulsions have been used in soil stabilization, particularly for the stabilization of sands in desert areas.

Some of the general properties of road emulsions are judged by the following tests :

(i) *Residue on Sieving* : It is desirable to see that not more than 0.25 percent by weight of emulsion consists of particles greater than 0.15 mm diameter.

(ii) *Stability to Mixing with Coarse Graded Aggregate* : This test carried out to find if the emulsion breaks down and coats the aggregate with bitumen too early before mixing is complete.

(iii) *Stability to Mixing with Cement* : This test is carried out to assess the stability of emulsions when the aggregate contains large proportions of fines.

(iv) *Water Cement* : To know the percentage water in the emulsion which depends on the type of the emulsion.

(v) *Sedimentation* : Some sedimentation may occur when a drum of emulsion is left standing before use, but on agitation, the emulsion redisperses and can be used.

(vi) *Viscosity* : The viscosity of emulsified bitumen should be low enough to be sprayed through jets or to coat the aggregates in simple mixing.

Three types of bituminous emulsion are prepared, viz., (i) Rapid Setting (RS), (ii) Medium Setting (MS) and (iii) Slow Setting (SS) types. Rapid Setting type emulsion is suitable for surface dressing and penetration macadam type of construction. Medium Setting type is used for premixing with coarse aggregates and Slow Setting type emulsion is suitable for fine aggregate mixes.

6.3.7 Tar

Tar is the viscous liquid obtained when natural organic materials such as wood and coal carbonized or destrictively distilled in the absence of air. Based on the material from which tar is derived, it is referred to as wood tar or coal tar; the latter is more widely used for road work because it is superior. Three stages for the production of road tar are :

(i) Carbonization of coal to produce crude tar

(ii) Refining or distillation of crude tar and

(iii) Blending of distillation residue with distillate oil fraction to give the desired road tar.

There are five grades of roads tars, viz., RT-1, RT-2, RT-3, RT-4 and RT-5, based on their viscosity and other properties. RT-1 has the lowest viscosity and is used for surface painting under exceptionally cold weather as this has very low viscosity. RT-2 is recommended for standard surface painting under normal Indian climatic conditions. RT-3 may be used for surface painting, renewal coats and premixing chips for top course and light carpets. RT-4 is generally used for premixing tar macadam in base course. For grouting purposes RT-5 may be adopted, which has the highest viscosity among the road tars.

The various tests that are carried out on road tars are listed below :

(i) Specific gravity test

(ii) Viscosity test on standard tar viscometer

(iii) Equiviscous temperature (EVT)

(iv) Softening point

(v) Softening point of residue

(vi) Float test

(vii) Water content

(viii) Distillation fraction on distillation upto 200°C, 200°C to 270°C and 270°C to 330°C.

(ix) Phenols, percent by volume

(x) Naphthalene, percent by weight

(xi) Matter insoluble in toluene, percent by weight

The requirements for the five grades of road tars based on the above test results are given by the ISI. The summary of the important test properties is given in Table 6.8.

Table 6.8 Properties and Requirements of Road Tars

Sl. No.	Property	Road Tar Grades				
		RT-1	RT-2	RT-3	RT-4	RT-6
1.	Viscosity by standard tar viscometer (10 mm)					
	(a) at temperature, °C	35	40	45	55	-
	(b) viscosity range, seconds	33-55	30-55	35-60	40-60	-
2.	Equiviscous Temperature (EVT) range, °C	32-36	37-41	43-46	53-57	63-67
3.	Softening point, °C	-	-	-	-	45-50
4.	Specific gravity range at 27°C	1.16-1.26	1.16-1.26	1.18-1.28	1.18-1.28	1.18-1.28

Comparison of Tar and Bitumen

Bitumen and tar have black to dark brown colour. But bitumen is a petroleum product whereas tar is produced by the destructive distillation of coal or wood. The chemical constituents of bitumen and tar are quite different. Bitumen is soluble in carbon disulphide and in carbon tetrachloride; but tar is soluble only in toluene. The coats aggregate more easily and retains it better in presence of water than bitumen. But the tar is considered to have much inferior weather resisting property. Tar is more temperature susceptible, resulting in a great variation in viscosity with temperature . Bitumens are less temperature susceptible. The free carbon content is more in tar as seen from the solubility test.

6.4 BITUMINOUS PAVING MIXES

6.4.1 Requirements of Bituminous Mixes

The mix design should aim at an economical blend, with proper gradation of aggregates and adequate proportion of bitumen so as to fulfil the desired properties of the mix. Bituminous concrete or asphaltic concrete is one of the highest and costliest type of flexible pavement layers used in the surfacing course. The desirable properties of a good bituminous mix are stability, durability, flexibility, skid resistance and workability.

Stability is defined as resistance of the paving mix to deformation under load and that it is a stress which causes a specified strain depending upon anticipated field conditions. Stability is a function of friction and cohesion. Frictional resistance is a function of both inter-particle friction and friction imparted by bituminous materials. Cohesion is mainly offered by the factors that influence the mass viscosity of bitumen binder. *Density* is

directly related to voids in the compacted mixture. Stability and density in general are correlated terms. If voids are restricted, the resulting strength property of the paving mixes improve. Minimum voids requirement qualified for a given mix should be so selected which would provide space for necessary densification that may develop under traffic movements and expansion of bitumen at high temperatures. In the absence of this, the bitumen *bleeds* over the surface and causes skidding.

Durability is defined as the resistance of the mix against weathering and abrasive actions. Weathering causes hardening and this depends upon loss of volatiles and oxidation. Tensile strain is introduced in the top layer consisting of bituminous mix when wheel loads ply over it. Excessive strain causes cracking or plastic failure.

Flexibility is a property of the mix that measures the level bending strength. Thus suitability of the given bituminous paving mix need the consideration of all the factors listed above. *Skid resistance* is defined as the resistance of the finished pavement against skidding and is a function of surface texture and bitumen content. Workability is the ease with which the mix can be laid and compacted. It is function of gradation of aggregates, their shape and texture, bitumen content and its type.

Mix design methods should aim at determining the properties of aggregates and bituminous material which would give a mix having the following properties.

(i) Sufficient stability to satisfy the service requirements of the pavement and the traffic conditions, without undue displacements.

(ii) Sufficient bitumen to ensure a durable pavement by coating the aggregate and bonding them together and also by water-proofing the mix.

(iii) Sufficient voids in the compacted mix as to provide a reservoir space for a slight amount of additional compaction due to traffic and to avoid flushing, bleeding and loss of stability.

(iv) Sufficient flexibility even in the coldest season to prevent cracking due to repeated application of traffic loads.

(v) Sufficient workability while placing and compacting the Mix.

(vi) The mix should be the most economical one that would produce a stable, durable and skid resistant pavement.

Three mix design methods, namely, Marshall, Hveem and Hubbard-Field methods are briefly explained in the following paragraphs. These methods have been widely used by various agencies in the design and construction with satisfactory results. In each of these methods, the laboratory test results on the mixes have been correlated with the performance studies in developing the design criteria.

6.4.2 Design of Bituminous Mixes

The following steps may be followed for a rational design of a bituminous mix.

(i) Selection of Aggregate

Aggregates which possess sufficient strength, hardness, toughness and soundness are chosen, keeping in view the availability and economic consideration. Crushed aggregates and sharp sands produce higher stability of the mix when compared with gravel and rounded sands.

(ii) Selection of Aggregate Grading

The properties of a bituminous mix including the density and stability are very much dependent on the aggregates and their grain size distribution.

Most of the agencies and engineering organisations have specified the use of densely graded mixes and they do not prefer the open grading. As higher maximum size of aggregate gives higher stability, usually the biggest size that can be adopted keeping in view of the compacted thickness of the layer is selected, provided all other factors are equal. In base course maximum aggregate size of 2.5 to 5 cm are used whereas for surface course 1.25 to 1.87 cm size are used in the mixes.

The gradation of final mix after blending of the aggregates and filler should be within the specified range. The gradation for 40 mm thick bituminous concrete surface course specified by the IRC is given in Table 6.9.

Table 6.9 Specified Gradation of Aggregates for Bituminous (Asphaltic) Concrete Surface Course

Sieve size, mm	Percent passing, by weight	
	Grade 1	Grade 2
20	-	100
12.5	100	80 – 100
10.0	80 – 100	70 – 90
4.75	55 – 75	50 – 70
2.36	35 – 50	35 – 50
0.600	18 – 29	18 – 29
0.300	13 – 23	13 – 23
0.150	8 – 16	8 – 16
0.075	4 – 10	4 – 10
Binder content, percent by weight of mix	5 – 7.5	5 – 7.5

(iii) Determination of Specific gravity

The specific gravity of the bituminous material is not usually determined, if already known. The specific gravity of aggregates are represented as either bulk specific gravity, or apparent specific gravity, or effective specific gravity. In bulk specific gravity the overall volume of the aggregate is taken. In apparent specific gravity the volume of capillaries which are filled by water on 21 hours soaking is excluded. When either bulk or apparent specific gravity is used, the specific gravity of such aggregate (i.e., coarse, fine and filler) is found and that of the combined aggregate is calculated. But when effective specific gravity is used, the specific gravity of the total or combined aggregate is determined and the average specific gravity G_a of blended aggregate mix is calculated from the equation,

$$G_a = \frac{100}{W_1/G_1 + W_2/G_2 + W_3/G_3 + W_4/G_4} \quad (6.9)$$

Where W_1, W_2, W_3, W_4 are percent by weight of aggregate 1, 2, 3 and 4; G_1, G_2, G_3 and G_4 are the specific gravities of the respective aggregates.

(iv) Proportioning of Aggregates

First the design grading is decided based on the type of the construction work, thickness of the layer and the availability of aggregates. Then the available aggregates

are proportioned either by analytical method, or graphical method, or by trial and error basis from laboratory test. Two of the graphical methods of proportioning, viz. triangular chart method and Rothfutch's method, are explained in Chapter 9.

(v) Preparation of Specimen

The preparation of specimen depends on the stability test method employed. Hence the size of the specimen, compaction and other specification should be followed as specified in the stability test method. The stability test methods which are in common use for the design mix are, Marshall, Hubbard-Field and Hveem. Hence after deciding the test methods, the specimens are moulded as per specification.

(vi) Determination of Specific Gravity of Compacted Specimen

The specific gravity of the compacted specimens, as moulded is determined. With known values of specific gravity of aggregate and bitumen, the theoretical maximum specific gravity of the mix G_t is calculated for a given bitumen content, using the formula,

$$G_t = \frac{100}{(100 - W_b)/G_a + W_b/G_b} \quad (6.10)$$

where W_b = percent by weight of bitumen content

G_b = specific gravity of bitumen

G_a = average specific gravity of aggregates.

The *theoretical density* γ_t, percent solids by volume is calculated from the formula.

$$\gamma_t = \frac{100\,G}{G_t}$$

Here, G = actual specific gravity of test specimen

G_t = theoretical maximum specific gravity

Hence *percent air voids* in the specimen,

$$V_v = 100 - \gamma_t = \frac{100\,(G_t - G)}{G_t}$$

The *Voids in the Mineral Aggregate* (VMA) is calculated from the equation :

$$\text{Volume VMA} = (V_v + V_b) = 100 - \frac{G}{W_a} \quad (6.11)$$

Here V_b is the percentage of bitumen and W_a is aggregate content percent by weight

Hence, *Percent Voids Filled with Bitumen* (VFB)

$$= \frac{100\,V_b}{VMA} \quad (6.12)$$

(vii) Stability Tests on Compacted Specimens

One of the stability tests is carried out based on the design method selected.

(viii) Selection of Optimum Bitumen Content

The optimum bitumen content is selected based on the test method adopted and the design requirements considered.

The Marshall, Hubbard-Field and Hveem stability test methods are explained here.

Marshall method of bituminous mix design

Bruce Marshall, formerly Bituminous Engineer with Mississippi State Highway Department formulated Marshall method for designing bituminous mixes. Marshall's test procedure was later modified and improved upon by U. S. Corps of Engineer through their extensive research and correlation studies. ASTM and other agencies have standardized the test procedure. Generally, this stability test is applicable to hot-mix design of bitumen and aggregates with maximum size 2.5 cm. In India, bituminous concrete mix is commonly designed by Marshall method.

In this method, the resistance to plastic deformation of cylindrical specimen of bituminous mixture is measured when the same is loaded at the periphery at a rate of 5 cm per minute. The test procedure is used in the design and evaluation of bituminous paving mixes. The test is extensively used in routine test programmes for the paving jobs. There are two major features of the Marshall method of designing mixes namely,

(i) density – voids analysis

(ii) stability – flow test

The stability of the mix is defined as a maximum load carried by a compacted specimen at a standard test temperature of 60°C. The flow is measured as the deformation in units of 0.25 mm between no load and maximum load carried by the specimen during stability test. (The flow value may also be measured by deformation units of 0.1 mm). In this test an attempt is made to obtain optimum binder content for the aggregate mix type and traffic intensity.

The apparatus consists of a cylindrical mould, 10.16 cm diameter and 6.35 cm height, with a base plate and collar. A compaction pedestal and hammer are used to compact a specimen by 4.54 kg weight with 45.7 cm height of fall. A sample extractor is used to extrude the compacted specimen from the mould. A breaking head is used to test the specimen by applying a load on its periphery perpendicular to its axis in a loading machine of 5 tonnes capacity at a rate of 5 cm per minute. A dial gauge fixed to the guide rods of the testing machine serves as flow meter to measure the deformation of the specimen during loading.

The coarse aggregates, fine aggregates and filler material should be proportioned and mixed in such a way that the final mix after blending has gradation within the specified range as given in Table 6.9. Approximately 1200 g of the mixed aggregates and the filler are taken and heated to a temperature of 175 to 190°C. The bitumen is heated to a temperature of 121 to 145°C and the required quantity of the first trial percentage of bitumen (say, 3.5 or 4.0 percent by weight of material aggregates) is added to the heated aggregates and thoroughly mixed at the desired temperature of 154 to 160°C. The mix is placed in a pre-heated mould and compacted by a rammer with 50 blows on either side at temperature of 138 to 149°C. (Suitable heating, mixing and compacting temperatures are chosen depending upon the grade of the bitumen). The weight of the mixed aggregate taken for the preparation of the specimen may be suitably altered to obtain a compacted

thickness of 63.5 ± 3.0 mm. Three or four specimens may be prepared using each trial bitumen content. The compacted specimens are cooled to room temperature in the moulds and then removed from the moulds using a specimen extractor. The diameter and mean height of the specimens are measured and then they are weighed in air and also suspended in water. The specimens are kept immersed in water in a thermostatically controlled water bath at 60 ± 1°C for 30 to 40 minutes. The specimens are taken out one by one, placed in the Marshall test head and tested to determine Marshall Stability Value which is the maximum load in kg before failure and the Flow Value which is the deformation of the specimen in 0.25 mm units upto the maximum load. The corrected Marshall Stability Value of each specimen is determine by applying the appropriate correction factor, if the average height of the specimen is not exactly 63.5 mm; the correction factors to be applied are given in Table 6.10.

Table 6.10 Correction Factors for Marshall Stability Values

Volume of specimen in cc	Thickness of specimen in mm	Correction factor
457 – 470	57.1	1.19
471 – 482	58.7	1.14
483 – 495	60.3	1.09
496 – 508	61.9	1.04
509 – 522	63.5	1.00
523 – 535	65.1	0.96
536 – 546	66.7	0.93
547 – 559	68.3	0.89
560 – 573	69.9	0.86

The above procedure is repeated on specimens prepared with other values of bitumen content, in suitable increments, say 0.5 percent, out about 7.5 or 8.0 percent bitumen by weight of total mix. The bulk density, percent air voids, voids in mineral aggregates and voids filled with bitumen are calculated using the following relationships.

Percent Air Voids

$$V_v = \frac{G_t - G_m}{G_m} \times 100 \tag{6.13}$$

Here G_m = bulk density or mass density of the specimen

G_t = theoretical specific gravity of mixture

$$G_t = \frac{1000}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_4}{G_4}}$$

where W_1 = percent by weight of coarse aggregate in total mix

W_2 = percent by weight of fine aggregate in total mix

W_3 = percent by weight of filler in total mix

W_4 = percent by weight of bitumen in total mix

G_1 = Apparent specific gravity of coarse aggregate

G_2 = Apparent specific gravity of fine aggregate

G_3 = Apparent specific gravity of filler

G_4 = Specific gravity of bitumen

Percent Voids in Mineral Aggregate (VMA)

$$VMA = V_v + V_b$$

Here V_v = volume of air voids, %

$$V_b = \text{volume of bitumen, \%} = G_m = \frac{W_4}{G_4}$$

Percent Voids Filled with Bitumen (VFB)

$$VFB = \frac{100\ V_b}{VMA} \qquad (6.14)$$

The average value of each of the above properties are found for each mix with the different bitumen contents. Graphs are plotted with the bitumen content on the X-axis and the following values on the Y-axis.

(i) Marshall stability value

(ii) Flow value

(iii) Unit weight

(iv) Percent voids in total mix (V_v)

(v) Percent voids filled with bitumen (VFB)

Typical plots of these are shown in Fig. 6.27.

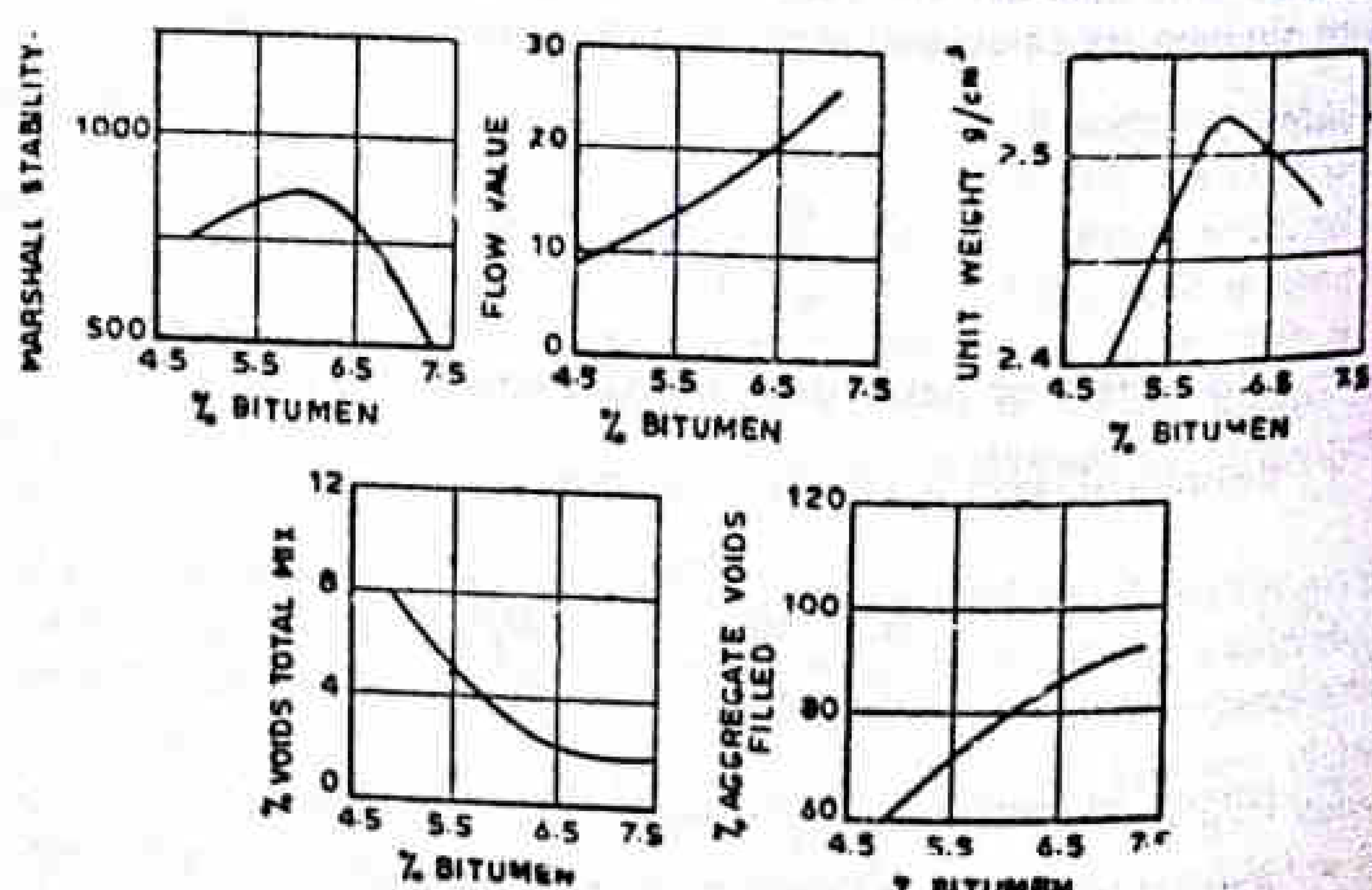


Fig. 6.27 Bituminous Mix Design by Marshall Test

The optimum bitumen content for the mix design is found by taking the average value of the following three bitumen contents found from the graphs of the test results.

(i) Bitumen content corresponding to maximum stability.

(ii) Bitumen content corresponding to maximum unit weight.

(iii) Bitumen content corresponding to the median of designed limits of percent air voids in total mix (4%).

The Marshall Stability value, Flow value and percent Voids Filled with Bitumen at the average value of bitumen content are checked with the Marshall mix design criteria/specifications, given in Table 6.11.

Table 6.11 Marshall Mix Design Criteria for Bituminous concrete

Test property	Specified value
Marshall Stability, kg	340 (minimum)
Flow value, 0.25 mm units	8 to 16
Air voids in total mix, V_v%	3 to 5
Voids filled with bitumen, VFB%	75 to 85

Mixes with very high Marshall stability values and low Flow values are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads, if the pavements components permit relatively high deflection values.

Example 6.3

Find the optimum bitumen content of a mix, considering the data plotted in Fig. 6.27.

Solution

Here the optimum bitumen contents corresponding to maximum values of stability and unit weight are 5.5 and 6 percent respectively, from Fig. 6.27.

Bitumen content corresponding to 4% air voids in total mix is 5.8 percent.

Hence the optimum bitumen content for the mix design is the average of the three values

$$= \frac{5.5 + 6 + 5.8}{3} = 5.8 \text{ percent}$$

The Flow value corresponding to 5.8 percent bitumen is 15, which is satisfactory as per Table 6.11. VFB at 5.8% bitumen is 78%, which is also as per the design criteria.

Modified Hubbard-Field method of bituminous mix design

The method was developed by *P. Hubbard and F. C. Field.* The original method was intended to design sheet asphalt mix. Later the method was modified for the design of bituminous mixes having coarse aggregate upto 19 mm size.

The equipment consists of 15.24 cm diameter mould and other compacting equipment including tampers and compression machine of capacity 5000 kg. There is a testing assembly consisting of a ring of internal diameter 14.6 cm through which the specimen is extruded by applying load through the compression machine. The assembly is shown in Fig. 6.28.

For the desired blend and gradation of the aggregates, batch weights are calculated for producing specimens of compacted size, 15.2 cm diameter and 7 to 7.6 cm height. The weighed aggregates, filter and the bituminous materials are heated to the prescribed

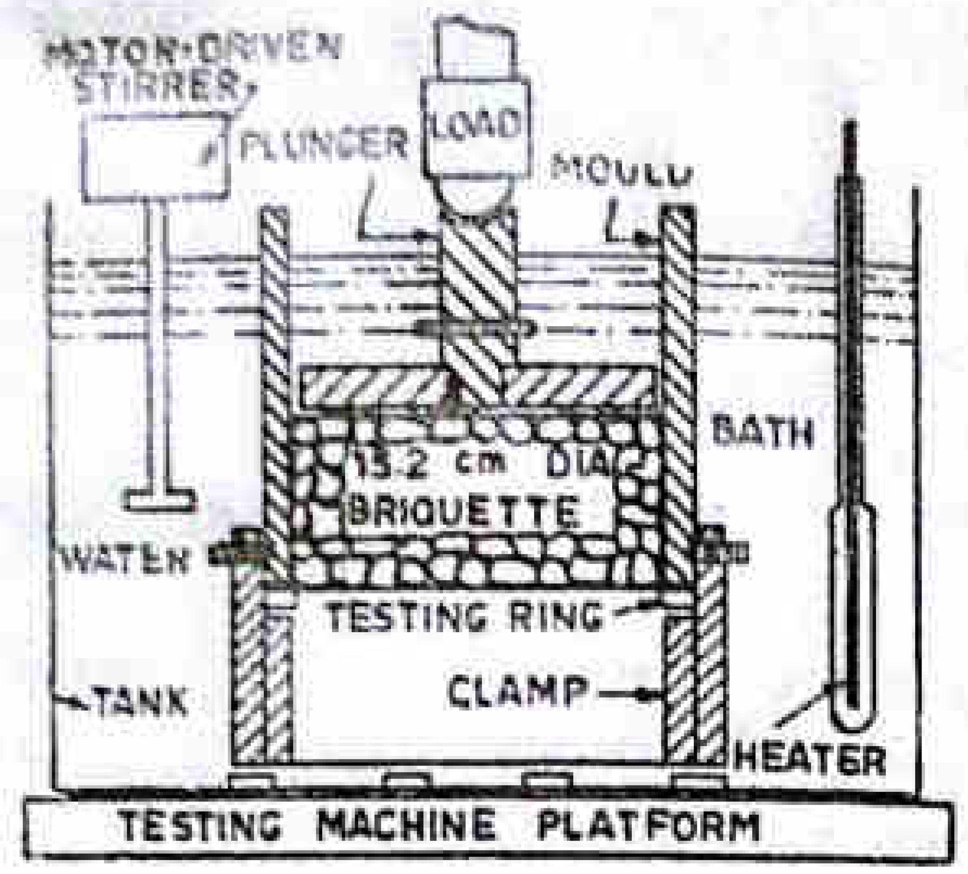


Fig. 6.28 Modified Hubbard-Field Test Set-up

temperature, mix placed in the preheated mould and tamped in two layers by 30 blows each with the specified tampers. The specimen is tamped again on the reverse side by 30 blows by each of the two tampers. Then a static load of 4536 kg is applied on the specimen for two minutes and the specimen is cooled in water to temperature less than 37.8°C, maintaining the same compressive load. The specimen is then removed, weighed and measured.

The specimen is placed in the test mould assembly over the test ring of internal diameter of 14.6 cm, and the plunger is loaded on the top of the specimen. The entire assembly is kept in a water bath maintained at 60°C for atleast one hour in position under the compression machine. The compressive load is applied at a constant rate of deformation of 6.1 cm per minute and the maximum load in kg developed during the test is recorded as the stability value. The average stability value of all the specimens tested using a particular mix is found. The tests are repeated with other bitumen contents as in Marshall method.

For each bitumen content the average value of specific gravity, percent voids in total mix and percent aggregate voids are calculated. The following graphs are plotted :

(i) Stability versus bitumen content

(ii) Unit weight versus bitumen content

(iii) Percent voids in total mix versus bitumen content.

(iv) Percent aggregate voids versus bitumen content.

Typical plots of these are shown in Fig. 6.29. The following criteria have been specified by the Asphalt Institute for the design of bituminous mix.

The property	Medium and light traffic	Heavy and very heavy traffic
Stability, kg	545 – 910	> 910
Voids, total mix, %	2 – 5	2 – 6

For determining the optimum bitumen content, first the bitumen content corresponding to 3 or 3.5 percent voids in total mix is found from the graph. The corresponding stability is read from the stability curve. If the stability values are within the specified limits, mix is satisfactory. If both stability and void requirements are not satisfied by a mix, the mix should be redesigned to correct the deficiency. The final selection of the mix design should be based on economics and suitability of the mix from the test requirements.

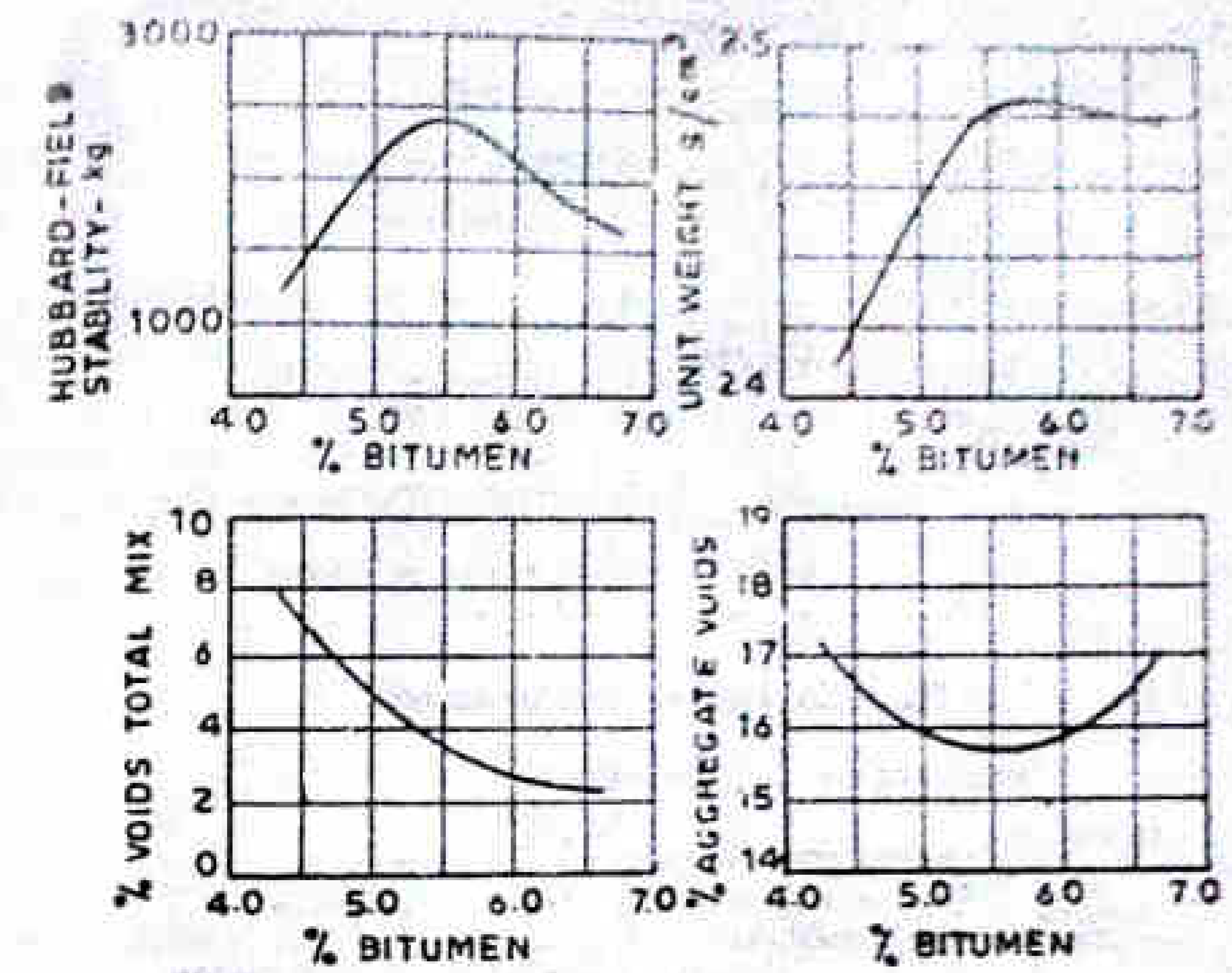


Fig. 6.29 Mix Design by Modified Hubbard Method

Example 6.4

Find the optimum bitumen content of the mix considering data plotted in Fig. 6.29.

Solution

Bitumen content corresponding to 3.5 percent voids in total mix is 5.5 percent. The Hubbard Field stability at 5.5 percent bitumen is 2300 kg and hence the mix is satisfactory for very heavy traffic.

Hveem method of bituminous mix design

This method was developed by *Francis N. Hveem*, Materials and Research Engineer for the California Division of Highways.

The equipment consists of compaction mould 10 cm inside diameter and 12.7 cm height. The mechanical compactor is a kneading compactor capable of exerting a force of 35 kg/cm^2 under the tamper foot. The swell test assembly consist of dial gauge and tripod. The Hveem stabilometer loading equipment and cohesiometer are shown in Fig. 6.30 and 6.31.

Stabilometer Test

The stabilometer consists of a cylindrical mould which can accommodate a specimen 10 cm diameter and 6.25 cm height resting over a rigid metal cylinder. The specimen is encased in the rubber membrane which acts as an inner wall of the mould. Fluid pressure

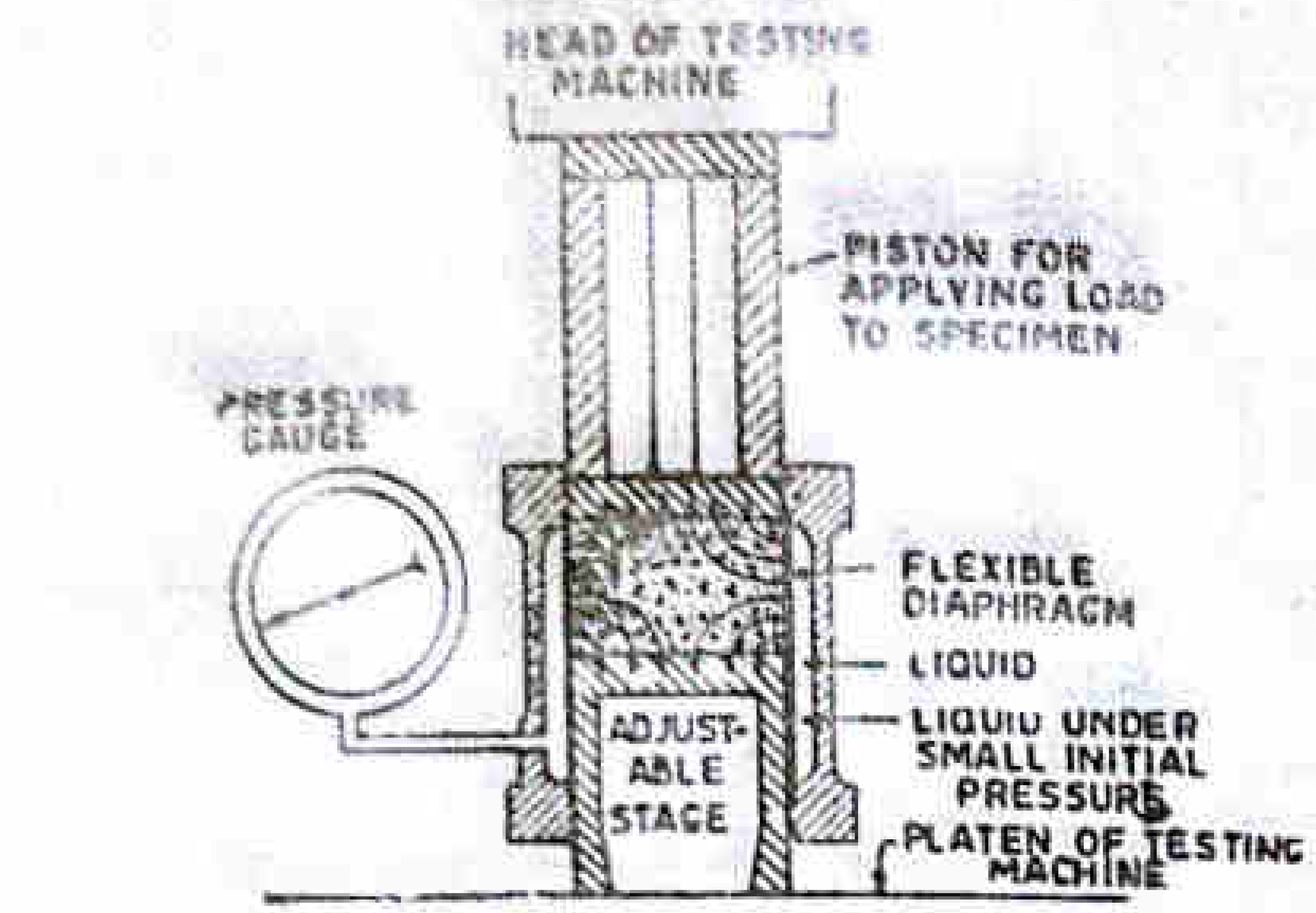


Fig. 6.30 Haveem Stabilometer

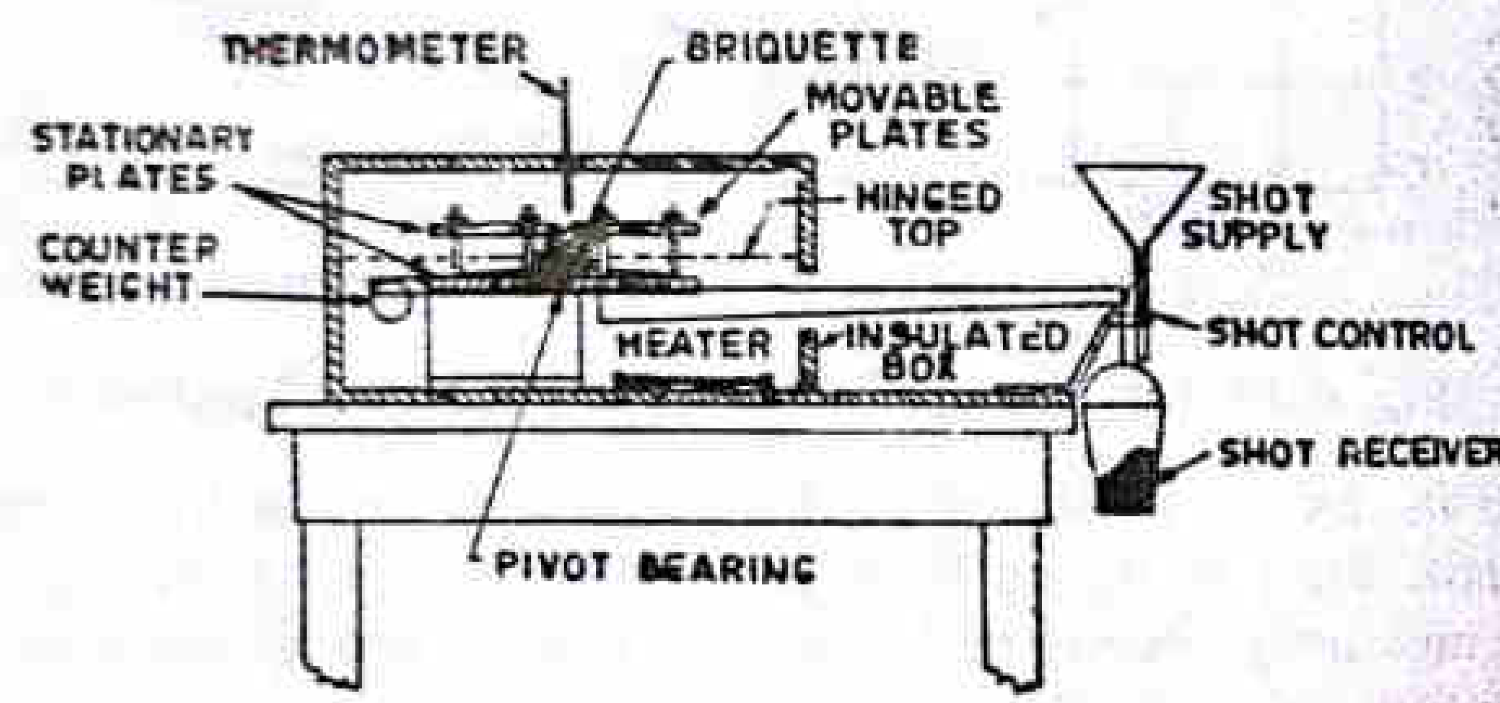


Fig. 6.31 Cohesiometer

can be applied through the membrane. Thus providing a lateral confinement to the specimen. The confining fluid pressure is applied by rotating a handle and is measured by a pressure gauge. The vertical pressure is applied through the loading head placed on the loading machine.

As per the requirement of the project, the aggregate gradation blend are chosen. The optimum bitumen content is then estimated using the *Centrifuge Kerosene Equivalent* (C.K.E.) method. (The percentage of kerosene retained in the aggregate after being soaked and centrifuged as specified is called C.K.E. value). Charts are available to find the optimum bitumen content from the C.K.E. value.

For mix design, specimens are prepared with three bitumen contents, one equal to 0.5 to 1.0 percent above and one 0.5 to 1.0 percent below the estimated optimum bitumen content. Two additional specimens are prepared using the estimated optimum bitumen content, for the swell tests. The specimens are compacted at 110°C using a kneading compactor, with a circular ram the pressure of which increases without impact upto 35 kg/cm^2, maintained for about 0.4 seconds and then released.

The compacted swell test specimens are cooled at room temperature for one hour and placed in the water pan for 24 hours after setting up the swell measuring dial arrangement. The swell is measured to the nearest 0.025 mm, and is represented as such.

The stabilometer test specimens are kept at temperature of 60°C and held in position in the Hveem Stabilometer. The fluid pressure is raised to 0.35 kg/cm^2 by the

displacement pump handle and the valve is closed. Vertical loads are then applied in sequence of 227, 454 and in increments of 454 kg up to a maximum of 2722 kg. The displacement valve is opened and the pressure is adjusted to 0.35 kg/cm^2.

The handle of the pump is rotated at two turns per second till the lateral pressure increases to 7 kg/cm^2 and the number of turns is noted.

The specimen from the stabilometer is recovered after releasing the pressures, weighed and measured to determine the bulk density. The sample is then maintained at a temperature of 60°C for two hours and placed in the cohesiometer, the cabinet of which is also maintained at a temperature of 60°C. The led shots are allowed to flow at a rate of 1800 g per minute until the specimen breaks, and the led shots in the bucket is weighed (L grams).

The stabilometer and cohesiometer value are calculated using the following formulae :

$$S = \frac{22.2}{\frac{P_h - D_2}{P_v - P_h} + 0.222} \qquad (6.15)$$

Here,

S = relative stability

P_v = vertical pressure at 28 kg/cm^2 or at a total load of 2268 kg.

P_h = horizontal pressure corresponding to P_v = 28 kg/cm^2.

D_2 = Displacement on specimen represented as number of turns of pump handle to raise P_h from 0.35 to 7 kg/cm^2.

$$C = \frac{L}{W\,(0.2H + 0.0176H^2)} \qquad (6.16)$$

Here

C = Cohesiometer value

L = weight of shots in gm

W = diameter or width of specimen in cm

H = height of specimen in cm

Using the specific gravity of the test specimens and the apparent specific gravity of aggregates the percent voids in the total mix is calculated.

The suitability of the hot mix is decided based on the design criteria given in Table 6.12.

Table 6.12 Design Criteria by Hveem Method

Test value	Criteria		
	Light traffic	Medium traffic	Heavy traffic
Stabilometer value, R	> 30	> 35	> 37
Cohesiometer value, C	> 50	> 50	> 50
Swell, mm	< 0.76	< 0.76	< 0.76
Air void, percent	> 4	> 4	> 4

The stabilometer is also employed in evaluating the resistance value (R-value) of soil subgrade material. The compaction is done using a kneading compactor with 24.6 kg per cm^2 pressure, 100 times. After the compaction, a load is applied at a rate of 907 kg per minute to record the exudation pressure required to force water out of the specimen. Expansion pressure is also noted permitting the specimen to remain in water for 16 to 20 hours.

The stabilometer resistance R-values is determined by placing the specimen in the stabilometer and applying the lateral and vertical pressures as specified. The R-value of soil is calculated from the formula :

$$R = 100 - \frac{100}{\frac{2.5}{D_2}\left(\frac{P_v}{P_h} - 1\right) + 1} \quad (6.17)$$

Here,

P_v = vertical pressure applied (11.2 kg/cm^2).

P_h = horizontal pressure transmitted at P_v = 11.2 kg/cm^2.

D_2 = displacement of stabilometer fluid necessary to increase the horizontal pressure from 0.35 to 7 kg/cm^2, measured in number of revolutions of the calibrated pump handle.

R-value of soil is used in the California Resistance value method of pavement design given in Art. 7.3.4.

6.5 PORTLAND CEMENT AND CEMENT CONCRETE

Portland cement is used in the construction of cement concrete pavements. Cement concrete pavements are considered to be the highest pavement type which withstand heavy traffic even under adverse subgrade and climatic conditions.

Portland cement is also being used in soil-cement stabilization for the construction of stabilized sub-base and base courses. The properties of cement and cement concrete and the mix design details of cement concrete are beyond the scope of this book and hence have not been included here.

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PROBLEMS

1. Why is it important for a highway engineer to study the behaviour of soil ? What are the desirable properties of subgrade soil ? Enumerate the identification and classification tests of soils.
2. Indicate the main features of Unified and H.R.B. classification systems. Discuss their advantages and limitations.
3. Discuss the suitability of the following soils as a subgrade material with particular reference to stability, volume change, drainage and frost action :

   GW, SM, SU, MH   CL, OH, A-1-a, A-2-7,
   A-3, A-4 (8), A-5 (11)   A-6 (3), A-7 (20)

4. The result of sieve analysis of a soil are given below.

Sieve size, mm	Percent passing
4.76	70
2.00	40
0.60	15
0.42	10
0.06	0

   (a) Classify the soil (i) Unified and (ii) HRB soil classification systems.

   (b) Discuss the suitability of the soil as a subgrade material.

5. The properties of a subgrade soil are given below

   Liquid limit = 75%

   Plastic limit = 55%

   Passing No. 200 sieve = 70%

   (a) Determine the Group Index and classify the soil by HRB soil classification system.

   (b) Discuss the suitability of the soil as a subgrade material.

6. What are the applications and limitations of shear, bearing and penetration tests.
7. Discuss the principles, applications and limitations of direct shear, triaxial and unconfined compression test.
8. Explain the plate bearing test procedure and how corrections for 'K' value may be made for a different plate size and for accounting for worst moisture conditions.
9. Explain CBR and the test procedure for laboratory and field tests. How are the results of the test obtained and interpreted ?
10. Explain the desirable properties of aggregate to be used in different types of pavement construction.
11. What are the various tests for judging the suitability of road stones ? Discuss the objects of carrying out each of these tests and their advantages and limitations.
12. Explain briefly the principle of the various tests on road stones; specify the desirable values of the test results.

13. What are the different types of bituminous materials used in road construction ? Under what circumstances each of these materials is preferred ?
14. Discuss the desirable properties of bitumen. Compare tar and bitumen
15. What are the various tests carried out on bitumen ? Briefly mention the principle and uses of each test.
16. List different types of cutbacks. When are these used ? Discuss in brief the tests carried out on cutback bitumen ?
17. Explain the uses of emulsion. How are they prepared ? Discuss in brief the tests carried out on emulsion ?
18. What are the desirable properties of bituminous mixes ? What are the steps in bituminous mix design ? Discuss briefly ?
19. Explain briefly the Marshall method of design.
20. Explain briefly the modified Hubbard Field method of bituminous mix design.
21. Explain briefly the Hveem method of bituminous mix design.

Chapter 7

Design of Highway Pavements

7.1 INTRODUCTION

7.1.1 Objects and Requirements of Pavements

The surface of the roadway should be stable and non-yielding, to allow the heavy wheel loads of road traffic to move with least possible rolling resistance. The road surface should also be even along the longitudinal profile to enable the fast vehicles to move safely and comfortably at the design speed. The earth road may not be able to fulfil any of the above requirements, especially during the varying conditions of traffic loads and the weather. At high moisture contents, the soil becomes weaker and soft and starts yielding under heavy wheel loads, thus increasing the tractive resistance. The unevenness and undulations of the surface along the longitudinal profile of the road causes vertical oscillations in the fast moving automobiles, increasing the fuel consumption and the wear of the vehicle components, resulting in a considerable increase in the vehicle operation cost. Apart from this uneven pavement surface causes discomfort and fatigue to the passengers of fast moving vehicles and cyclists. Therefore, in order to provide a stable and even surface for the traffic, the roadway is provided with a suitably designed and constructed pavement structure. Thus a pavement consisting of a few layers of pavement materials is constructed over a prepared soil subgrade to serve as a carriageway.

The pavement carries the wheel loads and transfer the load stresses through a wider area on the soil subgrade below. Thus the stresses transferred to the subgrade soil through the pavement layers are considerably lower than the contact pressure or compressive stresses under the wheel load on the pavement surface. The reduction in the wheel load stress due to the pavement depends both on its thickness and the characteristics of the pavement layers. A pavement layer is considered more effective or superior, if it is able to distribute the wheel load stress through a larger area per unit depth of the layer. However, there will be a small amount of temporary deformation even on a good pavement surface when heavy wheel loads are applied. One of the objectives of a well designed and constructed pavement is therefore to keep this elastic deformation of the pavement within the permissible limits, so that the pavement can sustain a large number of repeated load applications during the design life.

Based on the vertical alignment and the environmental conditions of the site, the pavement may be constructed over an embankment, cut or almost at the ground level itself. It is always desirable to construct the pavement well above the maximum level of the ground water to keep the subgrade relatively dry even during monsoons.

7.1.2 Types of Pavement Structure

Based on the structural behaviour, pavements are generally classified into two categories :

(i) Flexible pavements

(ii) Rigid pavements

Flexible pavements

Flexible pavements are those, which on the whole have low or negligible flexural strength and are rather flexible in their structural action under the loads. The flexible pavement layers reflect the deformation of the lower layers on-to the surface of the layer. Thus if the lower layer of the pavement or soil subgrade is undulated, the flexible pavement surface also gets undulated. A typical flexible pavement consists of four components : (i) soil subgrade (ii) sub-base course (iii) base course and (iv) surface course. (See Fig. 7.1 a).

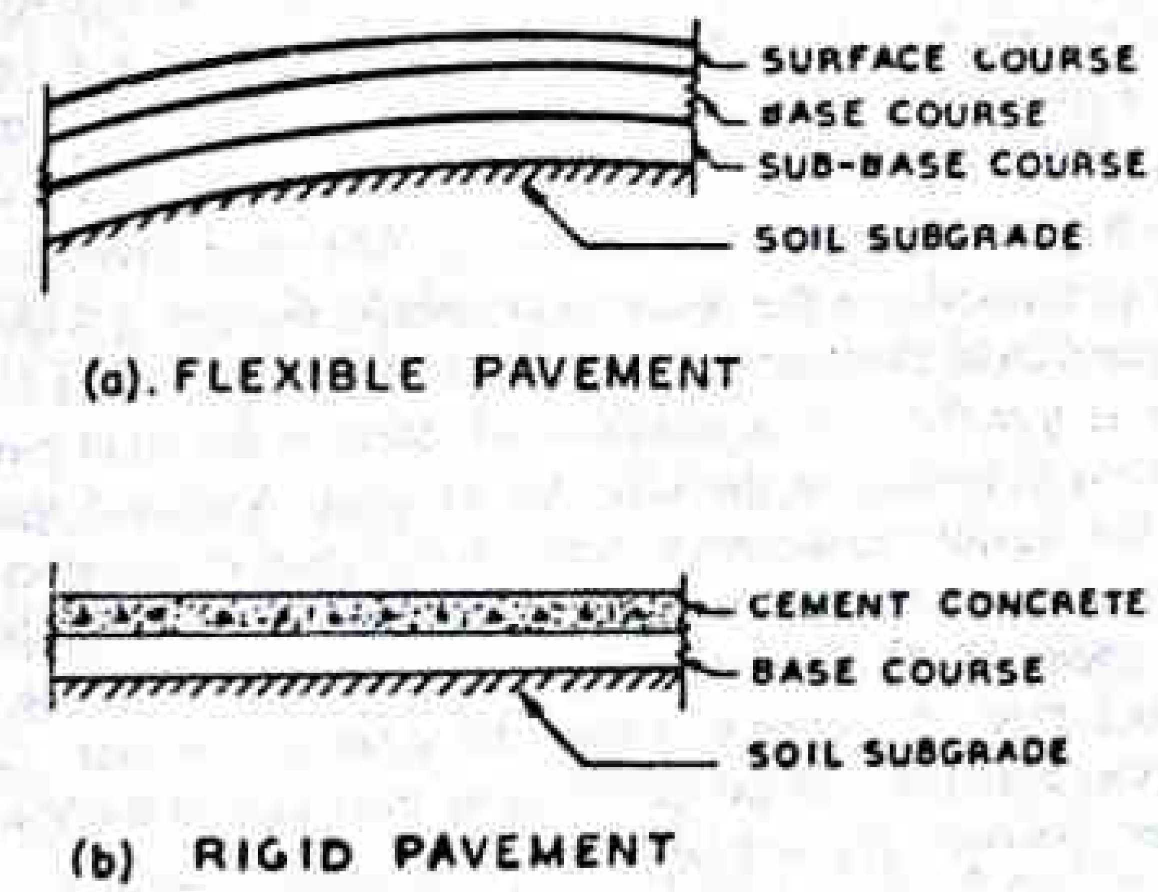


Fig. 7.1 Components of Flexible and Rigid Pavements

The flexible pavement layers transmit the vertical or compressive stresses to the lower layers by grain to grain transfer through the points of contact in the granular structure. A well compacted granular structure consisting of strong graded aggregate (interlocked aggregate structure with or without binder materials) can transfer the compressive stresses through a wider area and thus forms a good flexible pavement layer. The load spreading ability of this layer therefore depends on the type of the materials and the mix design factors. Bituminous concrete is one of the best flexible pavement layer materials. Other materials which fall under the group are, all granular materials with or without bituminous binder, granular base and sub-base course materials like the Water Bound Macadam, crushed aggregate, gravel, soil-aggregate mixes etc.

The vertical compressive stress is maximum on the pavement surface directly under the wheel load and is equal to the contact pressure under the wheel. Due to the ability to distribute the stresses to a larger area in the shape of a truncated cone, the stresses get decreased at the lower layers. Therefore by taking full advantage of the stress distribution characteristics of the flexible pavement, the *layer system concept* was developed. According to this, the flexible pavement may be constructed in a number of layer and the top layer has to be the strongest as the highest compressive stresses are to be sustained by this layer, in addition to the wear and tear due to the traffic. The lower layers have to take up only lesser magnitudes of stresses and there is no direct wearing action due to traffic loads, therefore inferior materials with lower cost can be used in the lower layers. The lowest layer is the prepared surface consisting of the local soil itself, called the subgrade. A typical cross section of a flexible pavement structure is shown in Fig. 7.1 (a); this consists of a wearing surface at the top, below which is the base course followed by the sub-base course and the lowest layer consists of the soil subgrade which has the lowest stability among the four typical flexible pavement components. Each of the flexible pavement layers above the subgrade, viz. sub-base, base course and the surface course may consist of one or more number of layers of the same or slightly different materials and specifications.

Flexible pavements are commonly designed using empirical design charts or equations taking into account some of the design factors. There are also semi-empirical and theoretical design methods.

Rigid pavements

Rigid pavements are those which possess note worthy flexural strength or flexural rigidity. The stresses are not transferred from grain to grain to the lower layers as in the case of flexible pavement layers. The rigid pavements are made of Portland cement concrete-either plain, reinforced or prestressed concrete. The plain cement concrete slabs are expected to take-up about 40 kg/cm^2 flexural stress. The rigid pavement has the slab action and is capable of transmitting the wheel load stresses through a wider area below. The main point of difference in the structural behaviour of rigid pavement as compared to the flexible pavement is that the critical condition of stress in the rigid pavement is the maximum flexural stress occurring in the slab due to wheel load and the temperature changes where-as in the flexible pavement it is the distribution of compressive stresses. As the rigid pavement slab has tensile strength, tensile stresses are developed due to the bending of the slab under wheel load and temperature variations. Thus the types of stresses developed and their distribution within the cement concrete slab are quite different. The rigid pavement does not get deformed to the shape of the lower surface as it can bridge the minor variations of lower layer.

The cement concrete pavement slab can very well serve as a wearing surface as well an effective base course. Therefore usually the rigid pavement structure consists of a cement concrete slab, below which a granular base or sub-base-course may be provided (see Fig. 7.1 b). Though the cement concrete slab can also be laid directly over the soil subgrade, this is not preferred particularly when the subgrade, consists of fine grained soil. Providing a good base or sub-base course layer under the cement concrete slab, increases the pavement life considerably and therefore works out more economical in the long run. The rigid pavements are usually designed and the stresses are analysed using the elastic theory, assuming the pavement as an elastic plate resting over an elastic or a viscous foundation.

Semi-rigid pavements

When bonded materials like the pozzolanic concrete (lime-flyash-aggregate mix), lean cement concrete or soil-cement are used in the base course or sub-base course layer the pavement layer has considerably higher flexural strength than the common flexible pavement layers. However these bonded materials do not possess as much flexural strength as the cement concrete pavements. Therefore when this intermediate class of materials are used in the base or sub-base course layer of the pavements, they are called semi-rigid pavements. This third category of semi-rigid pavements are either designed as flexible pavements with some correction factors to find the thickness requirements based on experience, or by using a new design approach. These semi-rigid pavement materials have low resistance to impact and abrasion and therefore are usually provided with flexible pavement surface course.

7.1.3 Functions of Pavement Components

Soil Subgrade and its Evaluation

The soil subgrade is a layer of natural soil prepared to receive the layers of pavement materials placed over it. The loads on the pavement are ultimately received by the soil subgrade for dispersion to the earth mass. It is essential that at no time, the soil subgrade is *overstressed*. It means that the pressure transmitted on the top of the subgrade is within the allowable limit, not to cause excessive stress condition or to deform the same beyond the elastic limit. Therefore it is desirable that atleast top 50 cm layer of the subgrade soil is well compacted under controlled conditions of optimum moisture content and maximum dry density. It is necessary to evaluate the strength properties of the soil subgrade. This helps the designer to adopt the suitable values of the strength parameter for design purposes and in case this supporting layer does not come upto the expectations, the same is treated or stabilized to suit the requirements.

Many tests are known for measuring the strength properties of the subgrades. Mostly the test are empirical and are useful for their correlation in the design. Some of the tests have been standardised for the use. The common strength tests for the evaluation of soil subgrade are :

California bearing ratio test

California resistance value test

Triaxial compression test and

Plate bearing test.

These tests have been explained in detail in Chapter 6 and in the book *Highway Materials Testing* by the authors.

California Bearing Ratio (CBR) test is a penetration test, evolved for the empirical method of flexible pavement design. The CBR test is carried out either in the laboratory on prepared specimens or in the field by taking in-situ measurements. This test is also carried out to evaluate the strength of other flexible pavement component materials.

California resistance value is found by using Hveem stabilometer. This test is used in an empirical method of flexible pavement design based on soil strength.

Though *triaxial test* is considered as the most important soil strength test, still the test is not very commonly used in structural design of pavements. This is because only a few theoretical methods make use of this triaxial test results.

The *plate bearing test* is carried out using a relatively large diameter plate to evaluate the load supporting capacity of supporting power of the pavement layers. The plate bearing test is used for determining the elastic modulus of subgrade and other pavement layers. The results of the plate bearing tests are used in flexible pavement design method like *McLeod* method and the method based on layer system analysis by *Burmister*. Also the test is used for the determination of modulus of subgrade reaction in rigid pavement analysis by *Westergaard's* approach.

Sub-base and Base Courses and their Evaluation

These layers are made of broken stones, bound or unbound aggregate. Some times in sub-base course a layer of stabilized soil or selected granular soil is also used. In some places boulder stones or bricks are also used as a sub-base or soling course. However at the sub-base course, it is desirable to use smaller size graded aggregates or soil-aggregate mixes or soft aggregates instead of large boulder stone soling course of brick on edge soling course, as these have no proper interlocking and therefore have lesser resistance to sinking into the weak subgrade soil when wet. When the subgrade consists of fine grained soil and when the pavement carries heavy wheel loads, there is a tendency for these boulder stones or bricks to penetrate into the wet soil, resulting in the formation of undulations and uneven pavement surface in flexible pavements. Sub-base course primarily has the similar function as of the base course and is provided with inferior materials than of base course. The functions of the base course vary according to type of pavement.

Base course and sub-base courses are used under flexible pavement primarily to improve the load supporting capacity by distributing the load through a finite thickness. Base courses are used under rigid pavement for :

(i) preventing pumping

(ii) protecting the subgrade against frost action.

Thus the fundamental purpose of a base course and sub-base course is to provide a stress transmitting medium to spread the surface wheel loads in such manner as to prevent shear and consolidation deformations.

The sub-base and base course layers may be evaluated by suitable strength or stability test like plate bearing, CBR or stabilometer test. Each test has its own advantages and limitations. Some times these layers are evaluated in terms of pressure distribution characteristics.

Wearing Course and its Evaluation

The purpose of the wearing course is to give a smooth riding surface that is dense. It resists pressure exerted by tyres and takes up wear and tear due to the traffic. Wearing course also offers a water tight layer against the surface water infiltration. In flexible pavement, normally a bituminous surfacing is used as a wearing course. In rigid pavements, the cement concrete acts like a base course as well as wearing course. There are many types of surface treatment employed as wearing course. The type of surface depends upon the availability of materials, plants and equipment and upon the magnitude of surface loads.

There is no test for evaluating the structural stability of the wearing course. However the bituminous mixes used in the wearing courses are tested for their suitability otherwise. The term stability is as associated with such evaluation. Most popular test in use is Marshall stability test wherein the optimum content of bitumen binder is worked out based on the stability density, VMA and VFB of the given gradings of the aggregate mixtures. The test has been discussed in article 6.4.2. Plate bearing test and *Benkelman Beam* test are also sometimes made use of for evaluating the wearing course and the pavement as a whole.

7.2 DESIGN FACTORS

7.2.1 Factors to be considered in Design of Pavements

Pavement design consists of two parts :

(i) mix design of materials to be used in each pavement component layer

(ii) thickness design of the pavement and the component layers.

The details of bituminous mix design are given in Chapter 6 and the design of soil-aggregate mixes and stabilized soil mixes are given in Chapter 9 of this book. The design factors and methods for the structural design of flexible and rigid pavements are presented in this chapter.

The various factors to be considered for the design of pavements are given below :

(i) Design wheel load

(ii) Subgrade soil

(iii) Climatic factors

(iv) Pavement component materials

(v) Environmental factors

(vi) Special factors in the design of different types of pavements.

The thickness design of pavement primarily depends upon the design wheel load. Higher wheel load obviously need thicker pavement, provided other design factors are the same. While considering the design wheel load, the effects of total static load on each wheel, multiple wheel load assembly (if any, like the dual or the dual-tandem wheel loads), contact pressure, load repetition and the dynamic effects of transient loads are to be taken into account. As the speed increases, the rate of application of the stress is also increased resulting in a reduction in the pavement deformation under the load; but on uneven pavements, the impact increases with speed. Some of the important design factors associated with the traffic wheel loads have been explained in the subsequent article.

The properties of the soil subgrade are important in deciding the thickness requirement of pavements. A subgrade with lower stability requires thicker pavement to protect it from traffic loads. The variations in stability and volume of the subgrade soil with moisture changes are to be studied as these properties are dependent on the soil characteristics. The stress-strain behaviour of the soil under static and repeated loads have also significance. Apart from the design, the pavement performance to a great extent depends on the subgrade soil properties and the drainage. The importance and desirable properties of subgrade soil have been explained in Art. 6.1.

Among the climatic factors, rain fall affects the moisture conditions in the subgrade and the pavement layers. The daily and seasonal variations in temperature has significance in the design and performance of rigid pavements and bituminous pavements. Where freezing temperatures are prevalent during winter, the possibility of frost action in the subgrade and the damaging effects should be considered at the design stage itself.

The stress distribution characteristics of the pavement component layers depend on characteristics of the materials used. The fatigue behaviour of these materials and their durability under adverse conditions of weather should also be given due consideration.

The environmental factors such as height of embankment and its foundations details, depth of cutting, depth of subsurface water table, etc. affect the performance of the pavement. The choice of the bituminous binder and the performance of the bituminous pavements depends on the variations in pavement temperature with the seasons in the region. The warping stresses in rigid pavements depend on the daily variations in temperature in the region and in the maximum difference in temperature between the top and bottom of the pavement slab.

In the case of semi-rigid pavement materials, the formation of shrinkage cracks, pattern and the mode of propogation and the fatigue behaviour under such adverse conditions of hair cracks are to be studied before arriving at a rational method of design for the semi-rigid pavements.

7.2.2 Design Wheel Load

The various wheel load factors to be considered in pavement design are :

(i) Maximum wheel load

(ii) Contact pressure

(iii) Dual or multiple wheel loads and equivalent single wheel load

(iv) Repetition of loads

Maximum wheel load

The wheel load configurations are important to know the way in which the loads of a given vehicle are applied on the pavement surface. Typical wheel load configuration of a tractor trailer unit of a heavy duty vehicle is shown in Fig. 7.2.

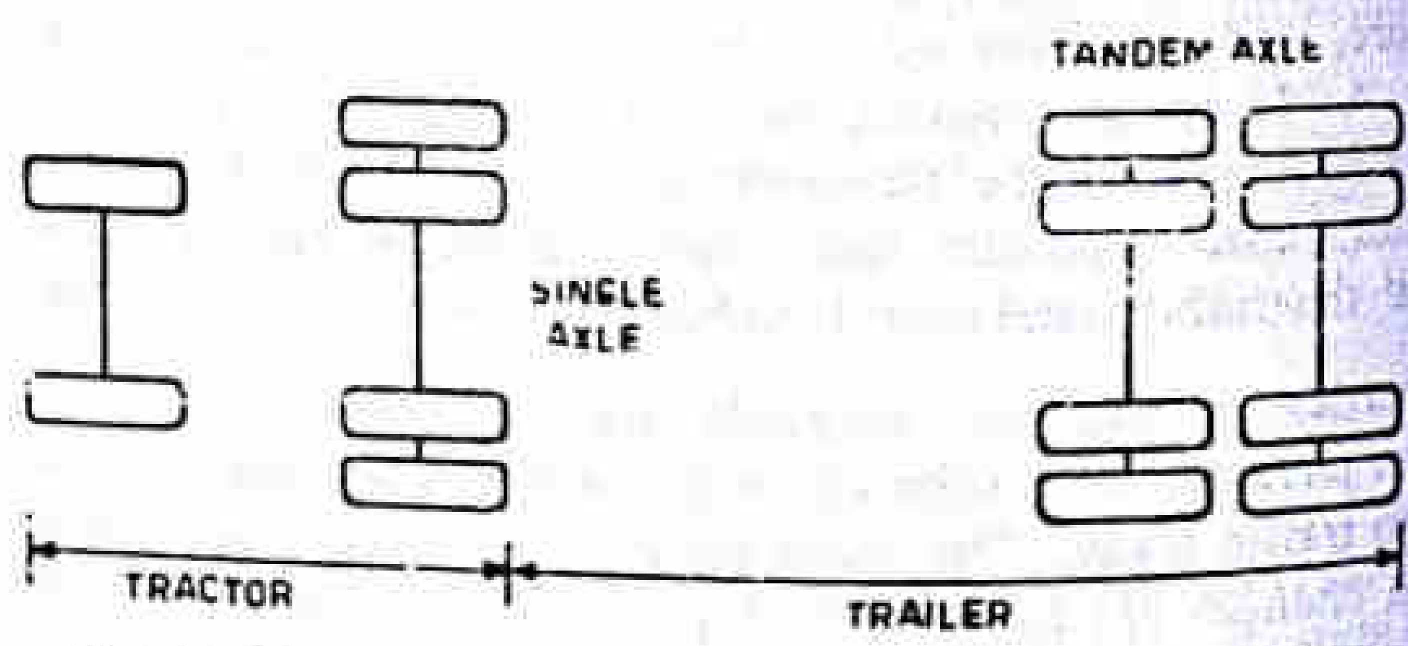


Fig. 7.2 Wheel Configuration of Tractor Trailer Unit

For highways the maximum legal axle load as specified by Indian Roads Congress is 8170 kg with a maximum equivalent single wheel load of 4085 kg. Total load influences the thickness requirements of pavements. Tyre pressure influences the quality of surface (wearing) course. In fact the magnitude of the vertical pressure at any depth of soil subgrade mass depends upon the surface pressure as well as on the total load.

The equation for vertical stress computations under a uniformly distributed circular load based on *Boussineq's* theory is given by :

$$\sigma_z = p\left[1 - \frac{z^3}{(a^2 + z^2)^{3/2}}\right] \qquad (7.1)$$

Here σ_z = vertical stress at depth z

p = surface pressure

z = depth at which σ_z is computed

a = radius of loaded area

Using the above equation the variation of vertical stress with depth is plotted as given in Fig. 7.3.

Contact pressure

As seen from the Fig. 7.3 the influence of tyre pressure is predominating in the upper layers. At a greater depth the effect of tyre pressure diminishes and the total load exhibits a considerable influence on the vertical stress magnitudes. Tyre pressure of high magnitudes therefore demand high quality of materials in upper layers in pavements. The total depth of pavement is however not influenced by the tyre pressure. With constant tyre pressure, the total load governs the stress on the top of subgrade within allowable limits.

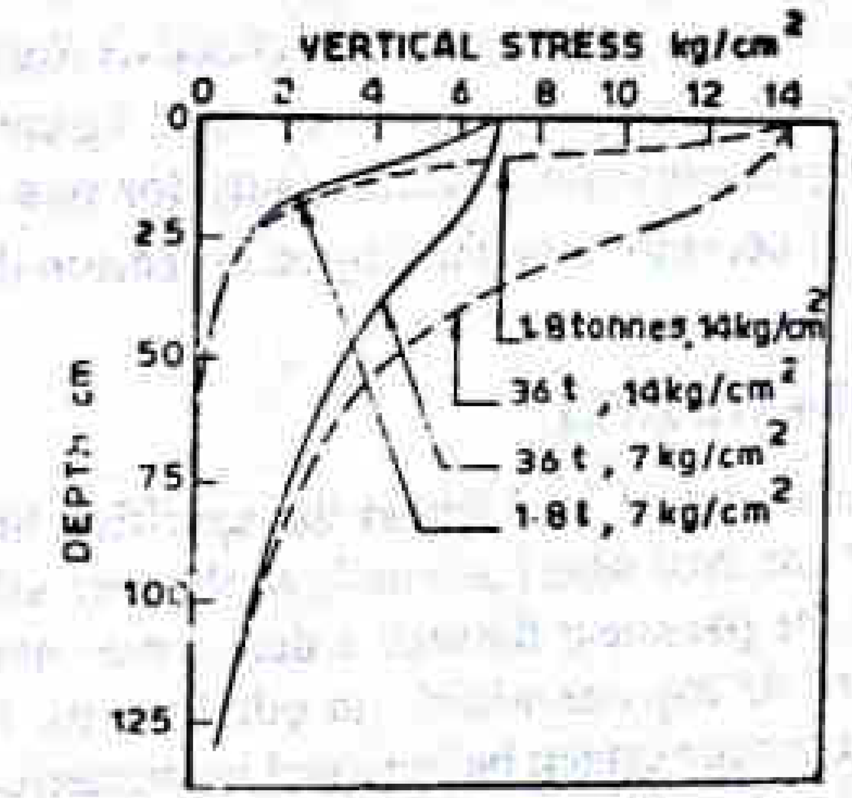


Fig. 7.3 Vertical Stress Distribution

The stresses on the pavement surface under the steel tyred wheels of bullock carts are very high. This demands use of very strong and hard aggregate for the wearing surface of the pavement. However the stresses at a lower layers of pavement due to the bullock cart wheel are negligibly small as the gross load is very small.

Generally the wheel load is assumed to be distributed over a circular area. But by measurement of the imprints of tyres with different load and inflation pressures, it is seen that contact areas in many cases are elliptical in shape. Three terms in use with reference to tyre pressure are :

Tyre pressure

Inflation pressure and

Contact pressure

Theoretically, all these terms should mean the same thing. Tyre pressure and inflation pressure mean exactly the same. The contact pressure is found to be more than tyre pressure when the tyre pressure is less than 7 kg/cm² and it is vice versa when the tyre pressure exceeds this value. Contact pressure can be measured by the relationship :

$$\text{Contact pressure} = \frac{\text{Load on wheel}}{\text{Contact area or area of imprint}}$$

The general variation between the tyre pressure and measured contact pressure is shown in Fig. 7.4.

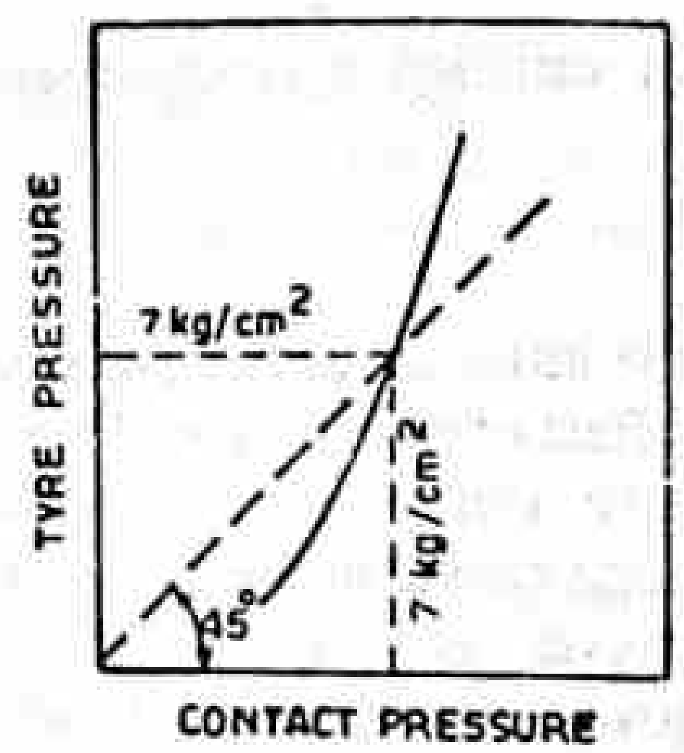


Fig. 7.4 Relationship between Tyre and Contact Pressure

The ratio of contact pressure to tyre pressure is defined as *Rigidity Factor*. Thus value of rigidity factor is 1.0 for an average tyre pressure of 7 kg/cm^2. This value is higher than unity for lower tyre pressures and less than unity for tyre pressures higher than 7 kg/cm^2. The rigidity factor depends upon the degree of tension developed in the walls of the tyres.

Equivalent single wheel load (ESWL)

To maintain the maximum wheel load within the specified limit and to carry greater load it is necessary to provide dual wheel assembly to the rear axles of the road vehicles. In doing so the effect on the pavement through a dual wheel assembly is obviously not equal to two times the load on any one wheel. In other words, the pressure at a certain depth below the pavement surface cannot be obtained by numerically adding the pressure caused by one wheel load. The effect is in between the single load and two times load carried by any one wheel. See Fig. 7.5. In order to simplify the analysis. The load dispersion is assumed to be at an angle of 45°. In the dual wheel load assembly, let d be the clear gap between the two wheels, S be the spacing between the centers of the wheels and a be the radius of the circular contact area of each wheel. Then S = (d + 2a).

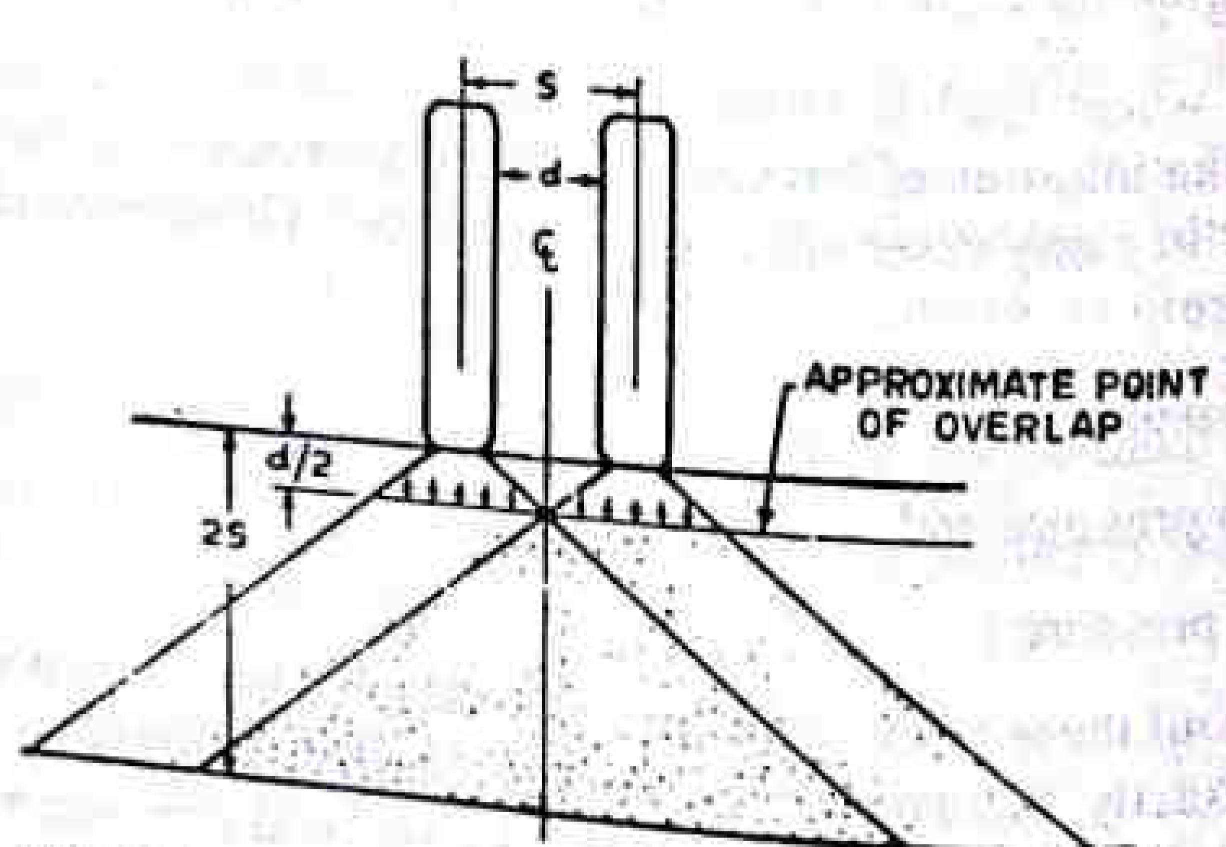


Fig. 7.5 Stress Overlap due to Dual Wheels

Upto the depth of d/2 each wheel load P acts independently and after this point the stresses induced due to each load begins to overlap. At depth 2S and above, the stresses induced are due to the effect of both wheels as the area of overlap is considerable. So the total stresses due to the dual wheels at any depth greater than 2S is considered to be equivalent to a single wheel load of magnitude 2P, though this stress is likely to be slightly greater than the stress due to the dual wheels.

Equivalent Single Wheel Load (ESWL) may be determined based on either equivalent deflection or equivalent stress criterion. Multiple wheel loads are converted to ESWL and this value is used in pavement design.

Suppose a dual wheel load assembly causes a certain value of maximum deflection Δ at a particular depth Z (say, depth equal to the thickness of the pavement). As per deflection criterion the ESWL is that single wheel load having the same contact pressure which produces the same value of maximum deflection at the depth Z. Similarly by stress criterion, the ESWL is the single wheel load producing the same value of maximum stress at the desired depth Z as the dual. The ESWL is usually determined by the equivalent stress criterion using a simple graphical method.

A straight line relationship is assumed between ESWL and depth on log-log scales. For determining ESWL the plot is made as shown in Fig. 7.6.

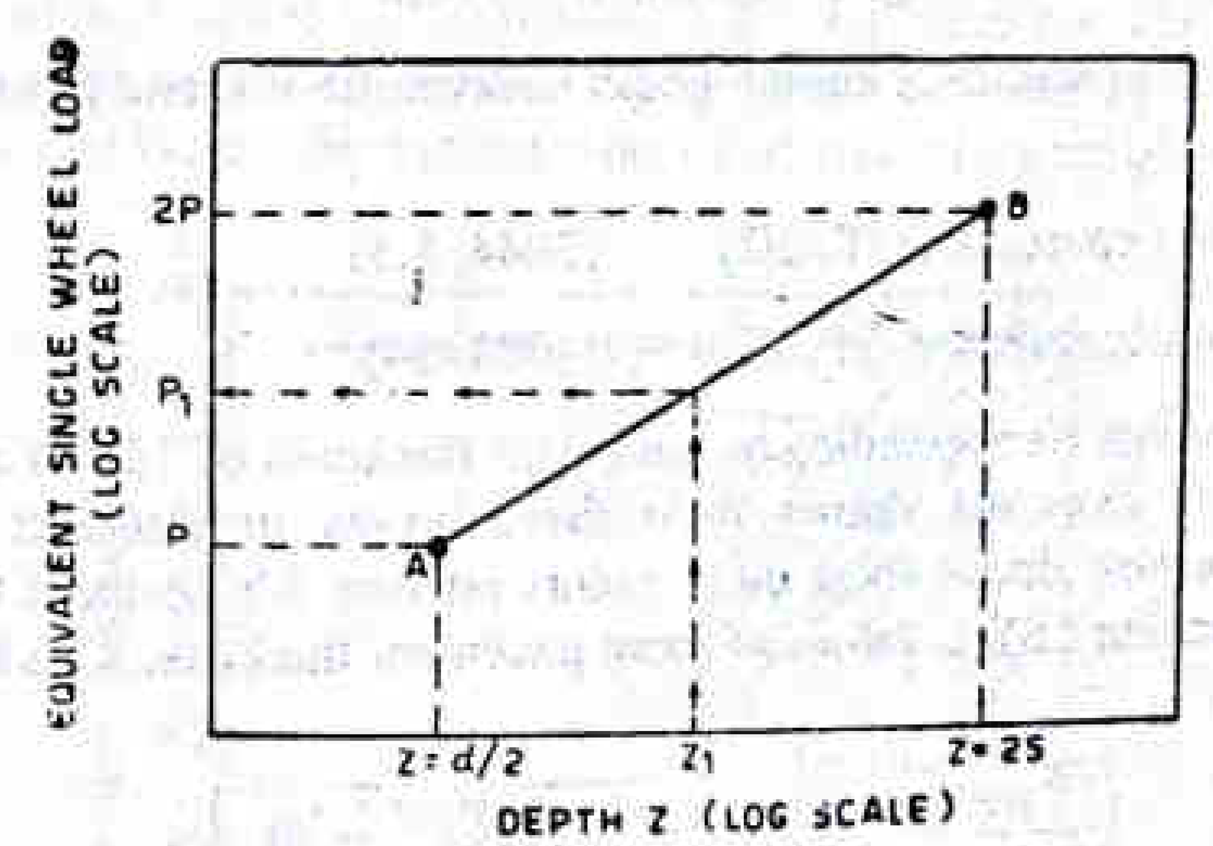


Fig. 7.6 Graphical Method for ESWL

Two points A and B are plotted on the log-log graph with coordinates of A (P, d/2) and B (2P, 2S). Line AB is a plot which is the locus of points where any single wheel load is equivalent to a certain set of dual wheels. To calculate the ESWL for a dual assembly, it is essential to estimate a design thickness of the pavement. Thus ESWL is obtained at the assumed thickness from this graph. The same is used in design calculations. If the design thickness so obtained is equal to the estimated thickness then the ESWL calculations could be considered as correct. Otherwise trials are made.

In heavy trucks and trailers, the load on each wheel may be further reduced by multiple wheels and tandem axles. Figure 7.2 shows an arrangement of dual wheels and tendem axles. It is possible to determine ESWL for such loading arrangements also.

Example 7.1

Calculate ESWL of a dual wheel assembly carrying 2004 kg each for pavement thickness of 15, 20 and 25 cms. Centre to centre tyre spacing = 27 cm and distance between the walls of the tyres = 11 cm.

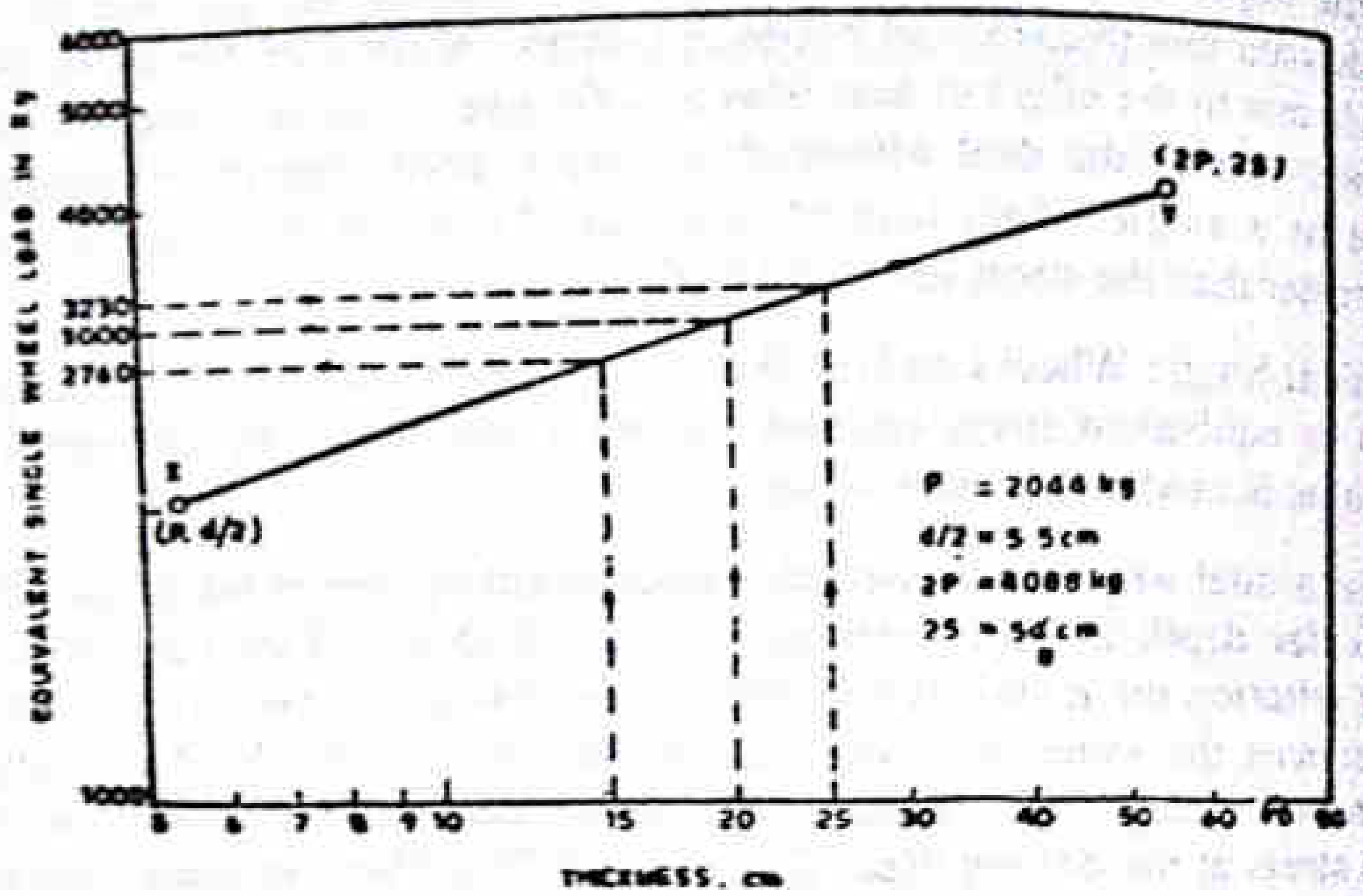


Fig. 7.7 ESWL Calculation (Example 7.1)

Solution

Here P = 2044 kg; 2P = 4088 kg; d = 11 cm; S = 27 cm

X and Y points are plotted on a log-log graph between ESWL and pavement thickness (See Fig. 7.7).

X has coordinates (P, d/2) = (2044, 5.5)

Y has coordinates (2P, 2S) = (4088, 54)

On the X-axis, points corresponding to pavement thickness of 15, 20 and 25 cms are marked and vertical lines are drawn from these points to intersect the line XY. Horizontal lines are now drawn from these points on line XY to meet the Y-axis, to obtain the corresponding ESWL values at these pavement thickness. ESWL values thus obtained are

Pavement thickness cm	ESWL kg
15	2760
20	3000
25	3230

Repetition of loads

The deformation of pavement or subgrade due to a single application of wheel load may be small. But due to repeated application of the load there would be increased magnitude of plastic and elastic deformations and the accumulated unrecovered or permanent deformations may even result in pavement failure.

It required to carry out traffic surveys for accounting the factor of repetitions for wheel loads in the design of pavement. Such data collected are converted to some constant equivalent wheel loads. Traffic composition in India is of mixed type and it is essential for design purposes to convert the various wheel loads to one single standard wheel load. Equivalent wheel load is a single load equivalent to the repeated applications of any particular wheel load on a pavement which requires the same thickness and strength of pavements.

If the pavement structure fails with N_1 number of repetitions of P_1 kg load and similarly if N_2 number of repetitions of P_2 kg load can also cause failure of the same pavement structure, then P_1N_1 and P_2N_2 are considered equivalent. *McLeod* has given a procedure for evolving *equivalent load factors* for designing flexible pavements.

McLeod assumes that the pavement thickness which are designed for a given wheel load would support one million repetition of such load during the life of pavement. For one load application, the pavement thickness so required is only one fourth the pavement thickness designed for 10^6 load repetitions.

For computing equivalent load factors, the plot similar to the one given in Fig. 7.8 was considered by McLeod. One fourth the design thickness were plotted for various wheel loads on vertical axis against one load application and total thickness (100%) were plotted on vertical axis drawn at 10^6 repetitions. The respective repetitions are then read from the figure for different loads at a pavement thickness of 25 cm (which is an average thickness for highway pavement on an ordinary soil subgrade). The values so obtained are given in Table 7.1. If the wheel load of 2268 kg (5000 lb) and the failure number of repetitions for 25 cm thick pavement are taken as standard, the number of failure repetitions for higher wheel loads may be obtained from Fig. 7.8. The number of failure repetitions for 2268 and 2722 kg are respectively 105,000 and 50,000 and so 2722 kg may be considered equivalent to 105,000/50,000 = 2.1 times the load value of 2268 kg. Hence the equivalent wheel load factor in this case is taken as 2.1 or say 2. The equivalent wheel load factors for various wheel load repetitions are given in Table 7.1.

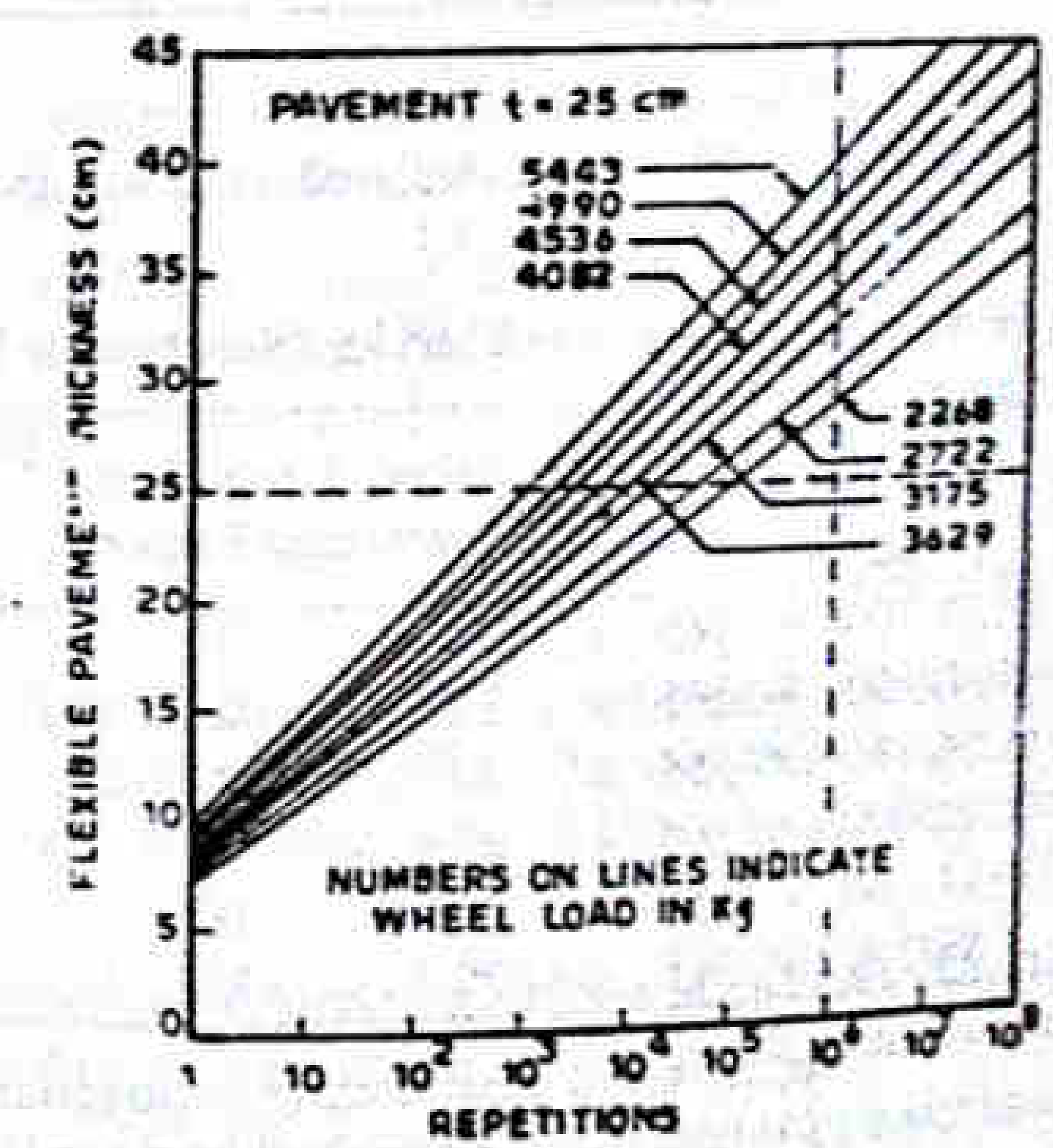


Fig. 7.8 Repetitions and Equivalent Load Factors

Equivalent load factors are employed to convert daily traffic count for each category of wheel load for design purposes.

Wyoming state Highway Department of USA employs the above concept for evaluating the design repetitions of wheel loads.

Table 7.1 Equivalent Wheel Load Factors

Wheel load kg	Repetitions to failure, number	Equivalent to 2268 kg	Equivalent load factors
2268	105,000	1.0	1
2722	50,000	2.0	2
3175	22,500	4.7	4
3629	13,000	8.2	8
4082	6,500	16.3	16
4536	3,300	32.0	32
4990	1,700	62.0	64
5443	1,000	105.0	128

Example 7.2

Calculate design repetitions for 20 years period for various wheel loads equivalent to 2268 kg wheel load using the following traffic survey data on a four lane road.

Wheel loads kg	Average Daily Traffic (both directions)	Percentage of total traffic volume
2268	Total volume (consideration traffic growth) 215	13.17
2722		15.30
3175		11.76
3629		14.11
4082		6.21
4536		5.84

Solution

Design repetitions for a period of 20 years calculated as given in Table 7.2. The equivalent load factors have been taken from Table 7.1

Table 7.2 Design Repetitions Equivalent to 2268 kg (Solution to Example 7.2)

Wheel loads kg	A.D.T. (both direction)	Percentage for each load	Days/ years	Number of years	Equivalent load Factors	Design repetitions equivalent of 2268 kg load
2268	215 ×	13.17/100 ×	365 ×	20	1	= 206,703
2722	215 ×	15.30/100 ×	365 ×	20	2	= 480,267
3175	215 ×	11.76/100 ×	365 ×	20	4	= 738,293
3629	215 ×	14.11/100 ×	365 ×	20	8	= 1,771,652
4082	215 ×	6.21/100 ×	365 ×	20	16	= 1,599,455
4536	215 ×	5.84/100 ×	365 ×	20	32	= 2,933,082
Total estimated repetitions (two directions)						= 7729,452
Design repetitions equivalents of 2268 kg wheel load per lane						= 7729,452/4
						= 19,32,363

7.2.3 Strength Characteristics of Pavement Materials

For design purposes, it is required that the various pavement materials are assigned strength parameters suitable to the design method employed for the purpose. Various materials used in sub-base course and base course are evaluated by different tests as indicated in article 6.1.8. The general strength values evaluated are :

(i) California Bearing Ratio (CBR) value

(ii) Elastic moduli

California Bearing ratio

The test has been explained in article 6.1.8. The strength values so obtained for the materials tested are of relative significance and do not provide as absolute measure. There are design methods which employ the CBR strength values of materials used in different pavement layers.

Elastic Modulii

Depending upon the design methods, the elastic modulii of different pavement materials are evaluated. Mainly, plate bearing test is employed for this purpose. This has been explained in article 6.1.8. Further modulus elasticity or modulus of deformation of highway materials may be determined from triaxial compression test.

The elastic modulii values of the following are determined by plate bearing tests :

(i) Subgrade modulus

(ii) Elastic moduli of base course and sub-base course materials.

Subgrade modulus is computed from the plate bearing test data. Boussinesq's settlement equation for maximum vertical deflection Δ at the surface and the centre of a flexible plate is given by :

$$\Delta = \frac{1.5\,p\,a}{E_s} \tag{7.2}$$

Here p is the uniform pressure on the flexible loaded plate of radius a. E_s is the modulus of elasticity of the soil.

If the load is applied by means of a rigid circular plate instead of flexible one, the pressure on the surface is not uniformly distributed and so the theoretical value of maximum deflection Δ at the surface in this case is given by :

$$\Delta = \frac{1.18\,p\,a}{E_s} \tag{7.3}$$

Plate bearing test conducted with a mild steel plate is considered relevant to the condition of *rigid plate* as in Eq. 7.3. But the wheel loads through inflate rubber tyres may be considered as *flexible plate* loading or loading with uniformly distributed pressure.

If the level of design deflection is defined, then from the plate bearing test carried out on a given soil subgrade with the plate of diameter = 2a, the pressure p can be recorded from the test plots. From Eq. 7.3,

$$\text{Subgrade modulus } E_s = 1.18\,\frac{p\,a}{\Delta}$$

Further extending the definition of subgrade modulus, *Westergaarde* employed the strength parameter of soil subgrade in rigid pavement analysis considering it as *modulus of subgrade reaction* K. The computation of this value from plate bearing test values is explained in article 6.1.8.

For computing elastic modulii of pavement materials, *Burmister's elastic layered system analysis* is employed. The displacement equations given by Burmister for a two layer system consisting of a pavement layer of thickness h with elastic modulus E_p laid over the subgrade is given by :

$$\Delta = 1.5 \frac{p\,a}{E_s} . F_2 \text{ (For flexible plate)} \quad (7.4)$$

$$\Delta = 1.18 \frac{p\,a}{E_s} . F_2 \text{ (For rigid plate)} \quad (7.5)$$

With known values of design deflections, yielded pressure p, subgrade modulus E_s and radius of loaded area a, the value of displacement factor F_2 is obtained. F_2 is a dimensionless factor and depends on the ratio of modulii of elasticity of subgrade to pavement E_s/E_p as well as the depth of radius ratio, z/a.

Thus using relationship between F_2 and the ratio of pavement thickness to radius of contact area, h/a the modulii ratio of subgrade and pavement material, E_s/E_p is calculated (See Fig. 7.22 given later). Since the value of E_s is known, the value of elastic modulus of the pavement Ep is thus obtained.

7.2.4 Climatic Variations

The climatic variations cause following major effects.

(i) Variation in moisture condition

(ii) Frost action

(iii) Variation in temperature

The pavement performance is very much affected by the variation in moisture and the frost. This is mainly because of the variation in stability and the volume of the subgrade soil due to these two effects. Variation in temperature generally affects the pavement materials like bituminous mixes and cement concrete.

Variation in Moisture Content

Considerable variations in moisture condition of subgrade soil are likely during the year, depending on climatic conditions, soil type ground water level and its variations, drainage conditions, type of pavement and shoulders. The surface water during rains may enter the subgrade either through the pavement edges or through the pavement itself, if it is porous. The subgrade moisture variations depend on fluctuations of ground water table. The moisture movement in subgrade is also caused by capillary action and vapour movement. However, high moisture variations could be controlled by providing suitable surface and sub-surface drainage system.

The stability of most of the subgrade soils are decreased under adverse moisture conditions. Presence of soil fraction with high plasticity will result in variations in volume (swelling and shrinkage) with variation in water content. As the moisture content of subgrade below the centre is often different from that at the pavement edges, there can be differential rise or fall of the pavement edges with respect to the centre, due to swelling and shrinkage of the subgrade soil. These effects are likely to cause considerable damages to the pavements and will also be progressive and cumulative.

Frost Action

Frost action refers to the adverse effective due to frost heave, frost melting or thaw and the alternate cycles of freezing and thawing. The frost action in general includes all effects associated with freezing temperature on pavement performance.

The held water in subgrade soil forms ice crystals at some spots if the freezing temperatures continue for a certain period. These ice crystals grow further in size if there is a continuous supply of water due to capillary action and the depressed temperature continues. This results in raising of portion of the pavement structure known as *frost heave*. If the frost heave cases uniform raising of pavement structure, the subgrade support is not adversely affected at this stage. However non-uniform heaving may cause damages.

Subsequent increase in temperature would result in melting or thawing of the frozen ice crystals and soften the road bed. The load carrying capacity of the subgrade is considerably decreased at this stage due to the voids created by the melted ice crystals and the excessive water trapped in the thawed soil below the pavement. Under heavy traffic, the pavement would deflect excessively causing progressive failure due to decreased load carrying capacity of the subgrade.

The freezing and thawing which occur alternately due to the variation in weather causes undulations and considerable damages to the pavement. Hence the overall effects due to frost heave, frost melting and alternate freeze-thaw cycles is called the *frost* action.

The various factors on which frost action depends may be broadly classified as :

(i) Frost susceptible soil

(ii) Depressed temperature below freezing point

(iii) Supply of water

(iv) Cover

The soil type, grain size distribution, permeability and capillarity of soil influence frost action. The temperature below freezing point and duration of the freezing temperature determines the depth up to which frost action exceeds. Unless there is a continuous supply of water, the small ice crystals formed can not grow in size. The supply of water may be from the ground water due to the capillary action or soil section. The rate of heat transfer depends on soil density and texture, moisture content and the proportion of frozen moisture in the soil mass under consideration. The type and colour of the cover affects the heat transfer from the atmosphere to the soil beneath the cover. For example temperature under a black top pavement will be higher than that under alight coloured pavement or base course.

One of the most effective and practical methods to decrease the damaging effects due to water and frost action is to install proper surface and subsurface drainage system. Construction of base, sub-base and top layer of subgrade, upto the desired depth, by granular and non-frost susceptible material with good drainage characteristics would go a long way in withstanding the adverse climatic conditions. Yet another effective method is by providing a suitable *capillary cut-off*. It is also possible to reduce the adverse effects of frost action on pavements by soil stabilization. The stabilized soil mix may be designed to withstand the adverse climatic conditions of alternate wet-dry and freeze-thaw cycles. Suitable stabilized soil mixes may be designed and provided for base course, sub-base courses and even at the top layer of subgrade. Salts like calcium chloride or sodium chloride when mixed with subgrade soil lowers the freezing temperature of the soil-water and hence temporarily decreases the intensity of frost action.

Variation in Temperature

Wide variation in temperature due to climatic changes may cause damaging effects in some pavements. Temperature stresses of high magnitude are induced in cement concrete pavements due to daily variations in temperature and consequent warping of the pavement as discussed in Article 7.4.3. Bituminous pavement become soft in hot weather and brittle in very cold weather.

From the above discussion, it is evident that the design and performance of pavements depend on the traffic loads, the subgrade, soil pavement materials and climatic conditions.

7.3 DESIGN OF FLEXIBLE PAVEMENTS

7.3.1 Flexible Pavement Design Methods

As discussed earlier, the flexible pavements are built with number of layers. In the design process, it is to be ensured that under the application of load none of the layers is overstressed. This means that at any instance no section of the pavement structure is subjected to excessive deformation to form a localized depression or settlement.

The maximum intensity of stresses occurs in the top layer of the pavement. The magnitude of load stresses reduces at lower layers. Hence the superior pavement materials are used in top layers of flexible pavements.

In the design of flexible pavements, it has yet not been possible to have a rational design method wherein design process and service behaviour of the pavement can be expressed or predicted theoretically by mathematical laws. Flexible pavement design methods are accordingly either empirical or semi-empirical. In these methods, the knowledge and experience gained on the behaviour of the pavements in the past are usefully utilized. The design methods therefore include methods based on soil classification like group index value and methods based on soil strength like California Bearing Ratio; California Resistance Value (R-value) and subgrade support based on plate bearing tests.

Besides these, the method based on stress-deformation characteristics of the pavement layers utilizing the theoretical considerations of elastic layered system analysis advocated by Burmister has considerable scope. An understanding of theoretical analysis by the designer is imperative since this helps to recognize the complexity of the phenomenon involved.

Various approaches of flexible pavement design may be thus classified into three broad groups.

(a) Empirical methods

(b) Semi-empirical or semi theoretical methods

(c) Theoretical methods

Empirical methods are either based on physical properties or strength parameters of soil subgrade. When the design is based on stress-strain function and modified based on experience, it may be called semi-empirical or semi-theoretical. There are design methods based on theoretical analysis and mathematical computations. Each one of the approaches has its own advantages and limitations.

Out of the various flexible pavement design methods available, the following are discussed here.

(i) Group Index method

(ii) California Bearing Ratio method

(iii) California R value or stabilometer method

(iv) Triaxial test method

(v) McLeod method

(vi) Burmister method

Of the design methods, the group Index, CBR, Stabilometer and McLeod methods are empirical methods. . The Triaxial test method is a theoretical method using empirical modifications as suggested by Kansas State Highway Department and therefore may be considered as a semi-empirical method. Burmister method is a theoretical approach using elastic two-layer theory.

7.3.2 Group Index Method

D. J. Steel in 1945 provided a discussion on the paper dealing with the Highway Research Board method of soil classification which included the suggested thickness requirements based on Group Index values. As discussed in article 6.1.6 the group index (GI) value is an arbitrary index assigned to the soil types in numerical equations based on the percent fines, liquid limit and plasticity index. Refer Eq. 6.1. The GI values of soils vary in the range of 0 to 20. The higher the GI value, weaker is the soil subgrade and for a constant value of traffic volume, the greater would be the thickness requirement of the pavement.

The design chart for Group Index method for determining the pavement thickness is given in Fig. 7.9. The traffic volume in this method is divided in three groups :

Traffic volume (commercial vehicles)	Number of vehicles per day
Light	Less than 50
Medium	50 to 300
Heavy	Over 300

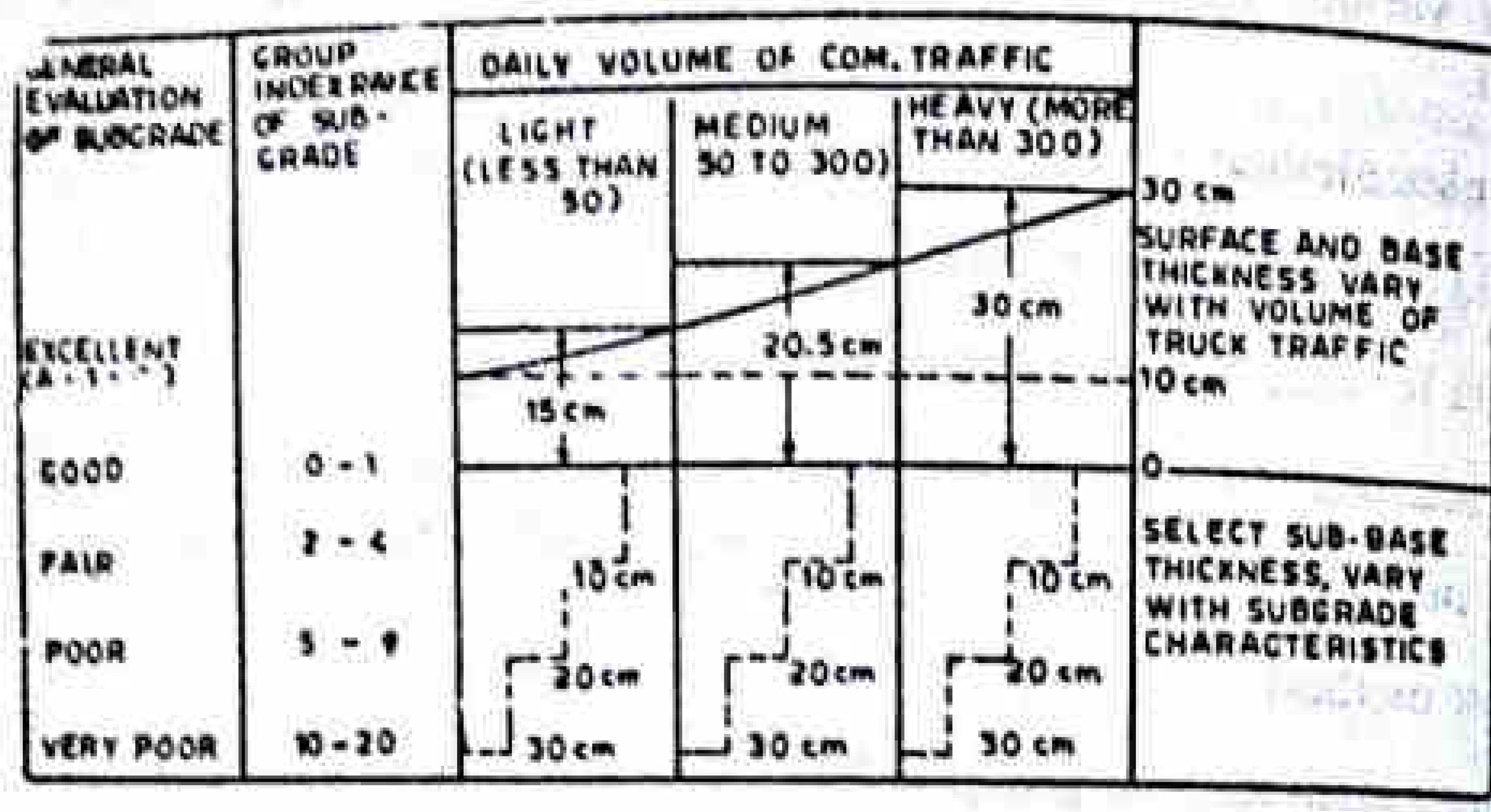
To design the pavement thickness by this method, first the GI value of the soil is found. The anticipated traffic is estimated and is designated as light, medium or heavy as indicated in Fig. 7.9-a. The appropriate design curve is chosen from Fig. 7.9-b and the total thickness of pavement (surface, base and sub-base course) is found from the Group Index design chart corresponding to the GI values of the soil.

Discussion

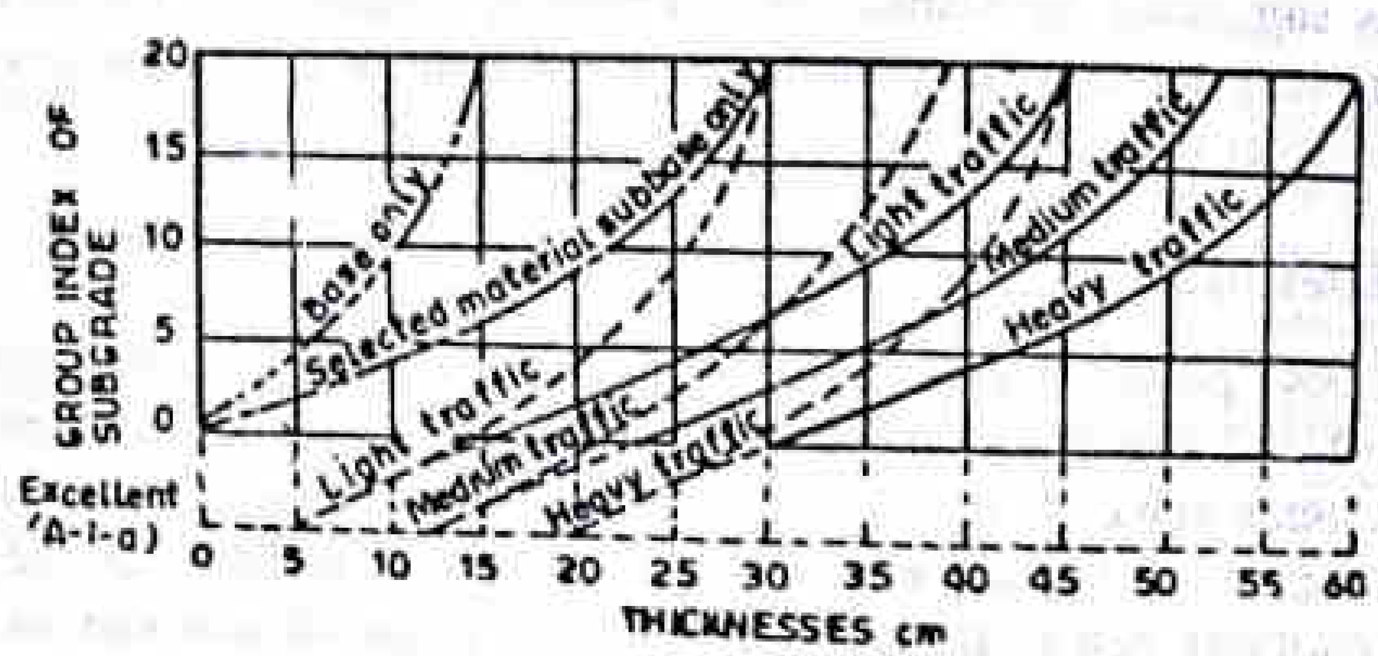
The GI method of pavement design is essentially an empirical method based on physical properties of the subgrade soil. This method does not consider the strength characteristics of the subgrade soil and therefore is open to question regarding the reliability of the design based on the index properties of the soil only. The Group Index method is illustrated by the following example.

Example 7.3

Soil subgrade sample collected from the site was analysed and the results obtained are as given below :



(a)



(b)

— Combined thickness of surface, base and sub-base

--- Thickness of surface and base.

Fig. 7.9 Design Chart by Group Index value

(i) Soil portion passing

0.074 mm sieve, percent = 50

(ii) Liquid Limit, percent = 40

(iii) Plastic Limit, percent = 20

Design the pavement section by group index method for the anticipated traffic volume of over 300 commercial vehicles per day.

Solution

The GI value could be calculated by either using Group Index charts vide Fig. 6.3 or by the Eq. 6.1.

From G I Chart

(i) Numerical value from Chart I for LL = 40 and percent passing 0.074 mm sieve = 50, is equal to 3.

(ii) Plasticity Index = LL − PL = 40 − 20 = 20. Numerical value from Chart II for PI = 20 and percent passing 0.074 mm sieve = 50, is equal to 3.5.

Total of value from Chart I + Chart II = 3 + 3.5 = 6.5 say GI = 7

From G I Equation

$$GI = 0.2a + 0.005\,a\,c + 0.01\,bd$$

Here

$$a = 50 - 35 = 15;\ b = 50 - 15 = 35$$

$$c = 40 - 40 = 0;\ d = 20 - 10 = 10$$

$$\text{GI value} = 0.2 \times 15 + 0 + 0.01 \times 35 \times 10 = 3 + 3.5 = 6.5 \text{ say } 7$$

Pavement Thickness Determination

The subgrade soil may be rated as poor from Fig. 7.9 (a) as the G.I. = 7. Traffic volume may be taken as heavy. The pavement layers may be designed either using Fig. 7.9 (a) or using the design chart given in Fig. 7.9 (b).

From Design Chart (Fig. 7.9 b)

(i) Thickness of sub-base for GI of 7 = 17 cm

(ii) Combined thickness of surface, base and sub-base course (using curve D for heavy traffic) = 47 cm

Hence thickness of base and surfacing = 47 − 17 = 30 cm

Discussion

It may be seen here that the quality of sub-base and base course materials is not considered in this method. The strength characteristics of the pavement materials also influence the thickness requirement. In this method, the emphasis is given only on subgrade soil type and certain physical properties of the soil.

California bearing ratio method

In 1928 California Division of Highways in the U.S.A. developed CBR method for pavement design. The majority of design curves developed later are based on the original curves proposed by *O. J. Porter*. At the beginning of the second World War, the *Corps Engineer* of USA made survey of the existing method of pavement design and adopted CBR method for designing military airport pavements. One of the chief advantages of CBR method is the simplicity of the test procedure. Details of CBR tests are explained in article 6.1.5.

The CBR tests were carried out by the California State Highway Department on existing pavement layers including subgrade, sub-base and base course. Based on the extensive CBR test data collected on pavement which behaved satisfactorily and those which failed, an empirical design chart was developed correlating the CBR value and the pavement thickness. The basis of the design chart is that a material with a given CBR required a certain thickness of pavement layer as a cover. A higher load needs a thicker pavement layer to protect the subgrade. Design curves correlating the CBR value with total pavement thickness cover were developed by the California State Highway Department for wheel loads of 3175 kg and 5443 kg representing light and heavy traffic. Later the design curve for 4082 kg wheel load was obtained by interpolation for medium traffic. The design curves are shown in Fig. 7.10.

Studies carried out by U. S. Corps of Engineers have shown that there exists a relationship between pavement thickness, wheel load, tyre pressure and C.B.R., value within a range of 10 to 12 percent. Therefore it is possible to extend the CBR design curves for various loading conditions, using the expression :

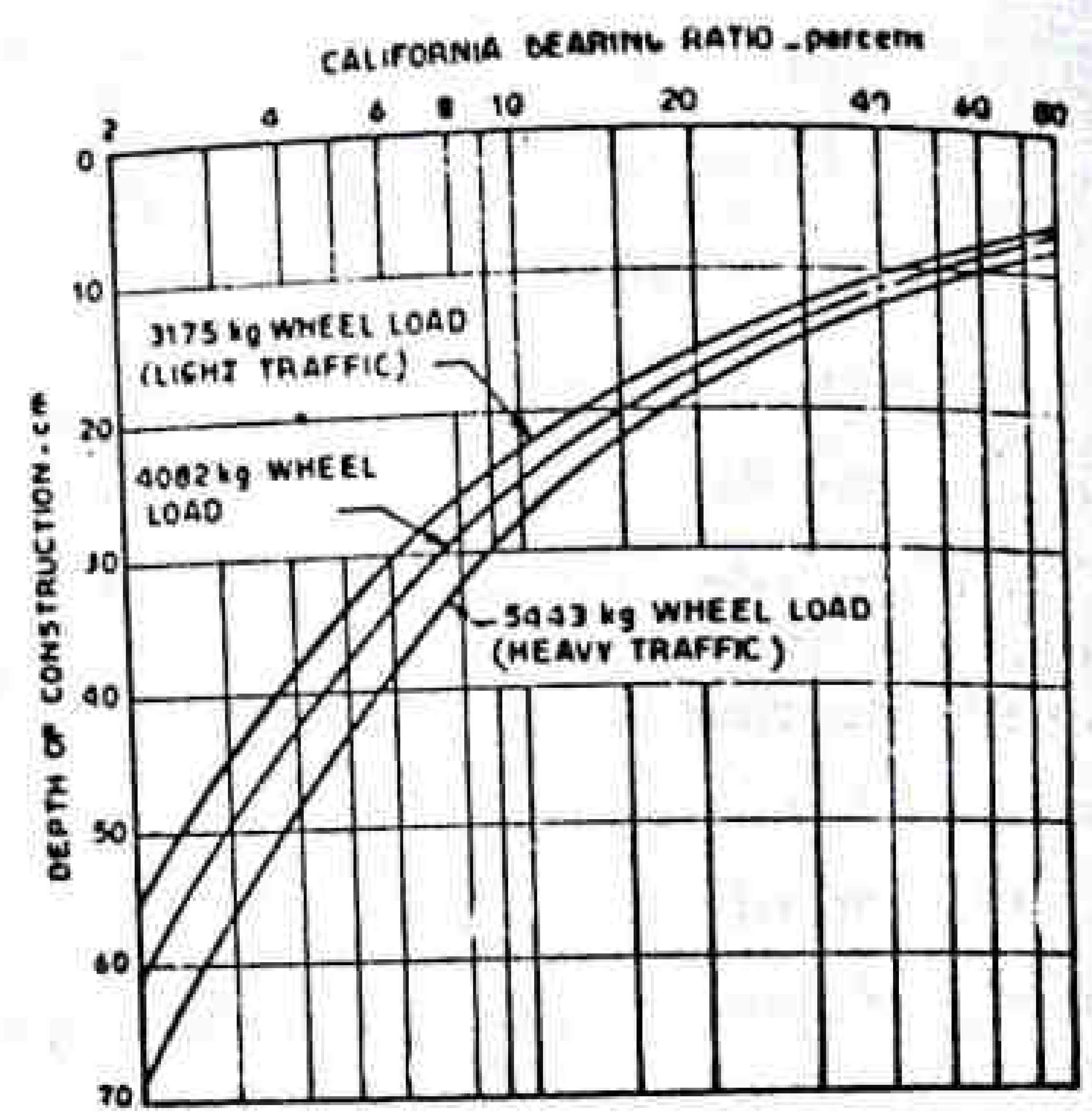


Fig. 7.10 Design Chart (California State Highway Department)

$$t = \sqrt{P}\left[\frac{1.75}{CBR} - \frac{1}{p\pi}\right]^{\frac{1}{2}} \qquad (7.6\text{-a})$$

$$t = \left[\frac{1.75\,P}{CBR} - \frac{A}{\pi}\right]^{\frac{1}{2}} \qquad (7.6\text{-b})$$

However these expressions are applicable only when the CBR value of the subgrade soil is less than 12 percent.

Here, t = pavement thickness, cm

P = wheel load, kg

CBR = California Bearing Ratio, percent

p = tyre pressure, kg/cm^2

A = area of contact, cm^2

The Indian Road Congress has recommended a CBR design chart for tentative use in India. Different curves A, B, C, D, E, F & G have been given based on the volume of commercial vehicles. See Fig. 7.11. This design chart is similar to the one followed in U.K.

Pavement Thickness Determination

In order to design a pavement by CBR method, first the soaked CBR value of the soil subgrade is evaluated. Then the appropriate design curve is chosen by taking the design wheel load as given in Fig. 7.10 or by taking the anticipated traffic into consideration (as given in Fig. 7.11). Thus the total thickness of flexible pavement needed to cover the subgrade of the known CBR value is obtained. In case there is a material superior than the soil subgrade, such that it may be used as sub-base course then the thickness of construction over this material could be obtained from the design chart knowing the CBR value of the sub-base. Thickness of the sub-base course is the total thickness minus the thickness over the sub-case.

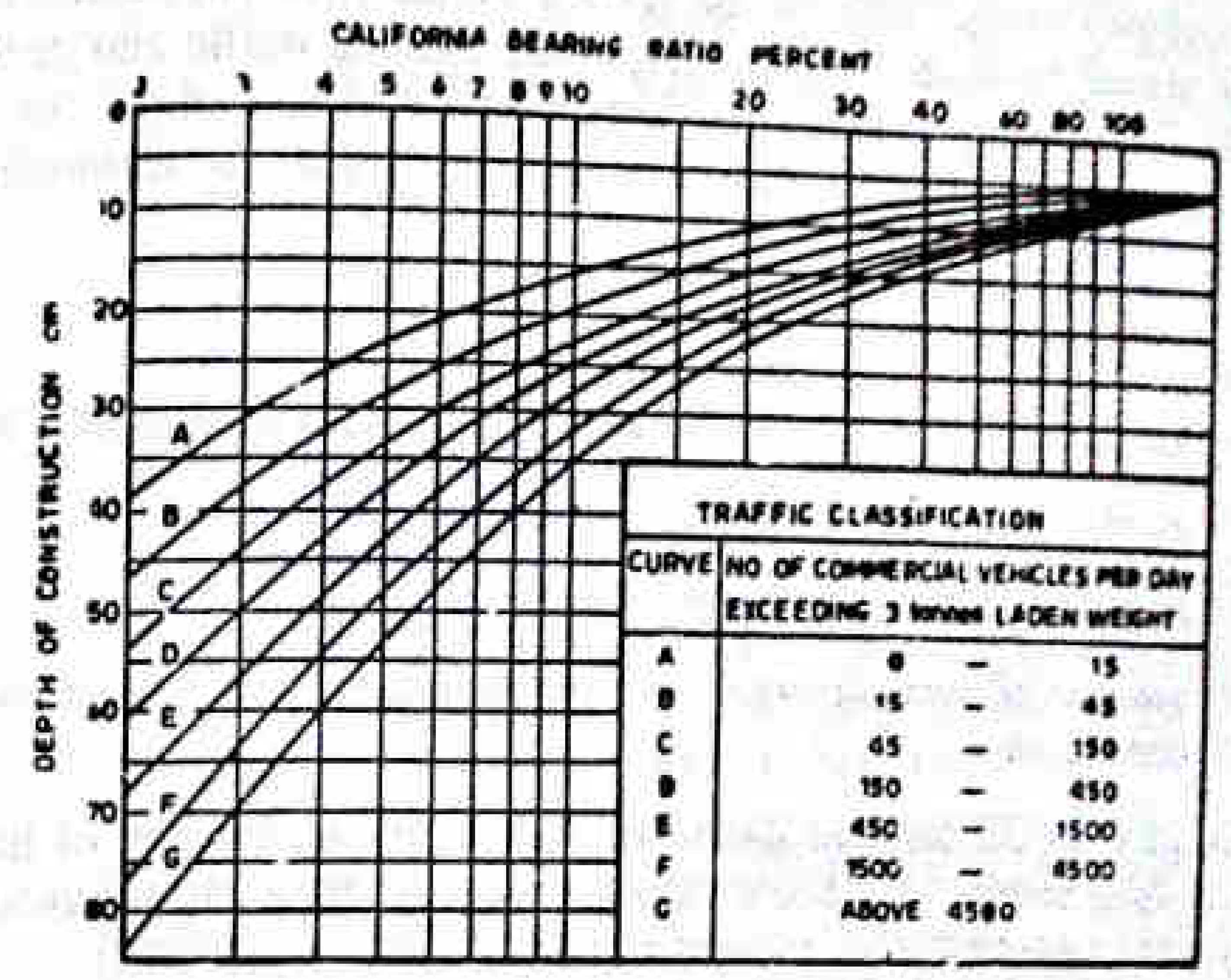


Fig. 7.11 C.B.R. Design Chart (Recommended by IRC)

Thus CBR method of flexible pavement design is based on strength parameter of subgrade soil and subsequent pavement material.

IRC Recommendations

Some of the important points recommended by the IRC for the CBR method of design (IRC : 37-1970) are given below :

(a) The CBR tests should be performed on remoulded soils in the laboratory. In-situ tests are not recommended for design purposes. The specimens should be prepared by static compaction wherever possible and otherwise by dynamic compaction. The standard test procedure should be strictly adhered to.

(b) For the design of new roads, the subgrade soil sample should be compacted at OMC to Proctor density whenever suitable compaction equipment is available to achieve this density in the field; otherwise the soil sample may be compacted to the dry density expected to be achieved in the field. In the case of existing roads, the sample should be compacted to field density of subgrade soil (at OMC or at a field moisture content).

(c) In new constructions the CBR test samples may be soaked in water for four days period before testing. However in areas with arid climate or when the annual rainfall is less than 50 cm and the water table is too deep to affect the subgrade adversely and when thick and impermeable bituminous surfacing is provided, it is not necessary to soak the soil specimen before carrying out CBR test. Wherever possible the most adverse moisture condition of the subgrade should be determined from the field study.

(d) Atleast three samples should be tested on each type of soil at the same density and moisture content. If the maximum variation in CBR values of the three specimens exceeds the specified limits, the design CBR should be the average of at least six samples. (The specified limits of maximum variation in CBR are 3% for CBR values upto 10% 5 for values 10 to 30 and 10% for values 30 to 60%).

(e) The top 50 cm of subgrade should be compacted atleast upto 95 to 100 percent of Proctor density.

(f) An estimate of the traffic to be carried by the road pavements at the end of expected life should be made keeping in view the existing traffic and probable growth rate of traffic. Pavements of major roads should be designed atleast for 10 years life period and the following formula may be used in such cases for estimating the design traffic.

$$A = P[1 + r]^{(n + 10)} \quad (7.6\text{-c})$$

where A = number of heavy vehicles per day for design (laden weight > 3 tonnes)

P = number of heavy vehicles per day at least count

r = annual rate of increase of heavy vehicles

n = number of years between the last count and the year of completion of construction.

The value of P in the formula should be the seven day average of heavy vehicles found from 24-hour counts. If reliable values of growth factor r is not available, a value of 7.5% may be assumed for roads in rural areas.

(g) The traffic for the design is considered in units of heavy vehicles (of laden weight exceeding 3 tonnes) per day in both directions and are divided into seven categories A to G. The suitable design curve should be chosen from the Table given in the design chart, (Fig. 7.11) after estimating the design traffic given in Eq. 7.6-c. The design thickness is considered applicable for single axle loads *upto* 8,200 kg and tandem axle loads upto 14,500 kg. For higher axle loads, the thickness values should be further increased.

(h) When sub-base course materials contain substantial proportion of aggregates of size above 20 mm, the CBR value of these materials would not be valid for the design of subsequent layers above them. Thin layers of wearing course such as surface dressing or open graded premixed carpet upto 2.5 cm thickness should not be counted towards the total thickness as they do not increase the structural capacity as the pavement.

Example 7.4

The CBR value of subgrade soil is 5%, calculate total thickness of a pavement using

(i) design curve developed by California State Highway Department

(ii) design chart recommended by IRC

(iii) design formula developed by the US Crops of Engineers

Assume 4100 kg wheel load or medium light traffic of 200 commercial vehicles per day for design.

$$\text{Tyre pressure} = 6 \text{ kg/cm}^2$$

Solution

(i) Using the design chart of California State Highway Department, the pavement thickness for 4100 kg wheel load and CBR = 5% (See Fig. 7.9) = 38 cm.

(ii) Using the design chart recommended by IRC (see Fig. 7.10) for 200 commercial vehicles per day and using curve D and for CBR value = 5% the thickness = 37.5 cm.

(iii) Using design formula given in Eq. 7.6-a,

$$t = \sqrt{P}\left[\frac{1.75}{CBR} - \frac{1}{p\pi}\right]^{\frac{1}{2}}$$

$$P = 4100 \text{ kg}$$

$$p = 6 \text{ kg/cm}^2$$

$$t = \sqrt{4100}\left[\frac{1.75}{5} - \frac{1}{6\pi}\right]^{\frac{1}{2}} = 35.5 \text{ cm}$$

Example 7.5

Soil subgrade sample was obtained from the project site and the CBR tests was conducted at field density. The following were the results :

Penetration mm	Load kg	Penetration mm	Load kg
0.0	0.0	3.0	56.5
0.5	5.0	4.0	67.5
1.0	16.2	5.0	75.2
1.5	28.1	7.5	89.0
2.0	40.0	10.0	99.5
2.5	48.5	12.5	106.5

It is desired to use the following materials for different pavement layers.

(i) Compacted sandy soil with 7 percent CBR

(ii) Poorly graded gravel with 20 percent CBR

(iii) Well graded gravel with 95 percent CBR

(iv) Minimum thickness of bituminous concrete surfacing may be taken as 5 cm

The traffic survey revealed the present ADT of commercial vehicle as 1200. The annual rate of growth of traffic is found to be 8 percent. The pavement construction is to be completed in three years after the last traffic count.

(a) Design the pavement section by CBR method as recommended by IRC, using all the four pavement materials.

(b) Suggest alternate design without using poorly graded gravel.

Discuss the limitation of CBR method of pavement design in the light of the above results.

Solution

CBR Value of Soil Subgrade

The plot is made between load in kg versus penetration of plunger for the test data obtained for soil subgrade as given in Fig. 7.12. Loads at 2.5 and 5.0 mm penetration (after correction) are 55 and 78 kg respectively.

$$\text{Area of plunger of dia 5 cm} = 19.6 \text{ cm}^2$$

$$\text{Pressure at 2.5 mm penetration} = \frac{55}{19.6} \text{ kg/cm}^2$$

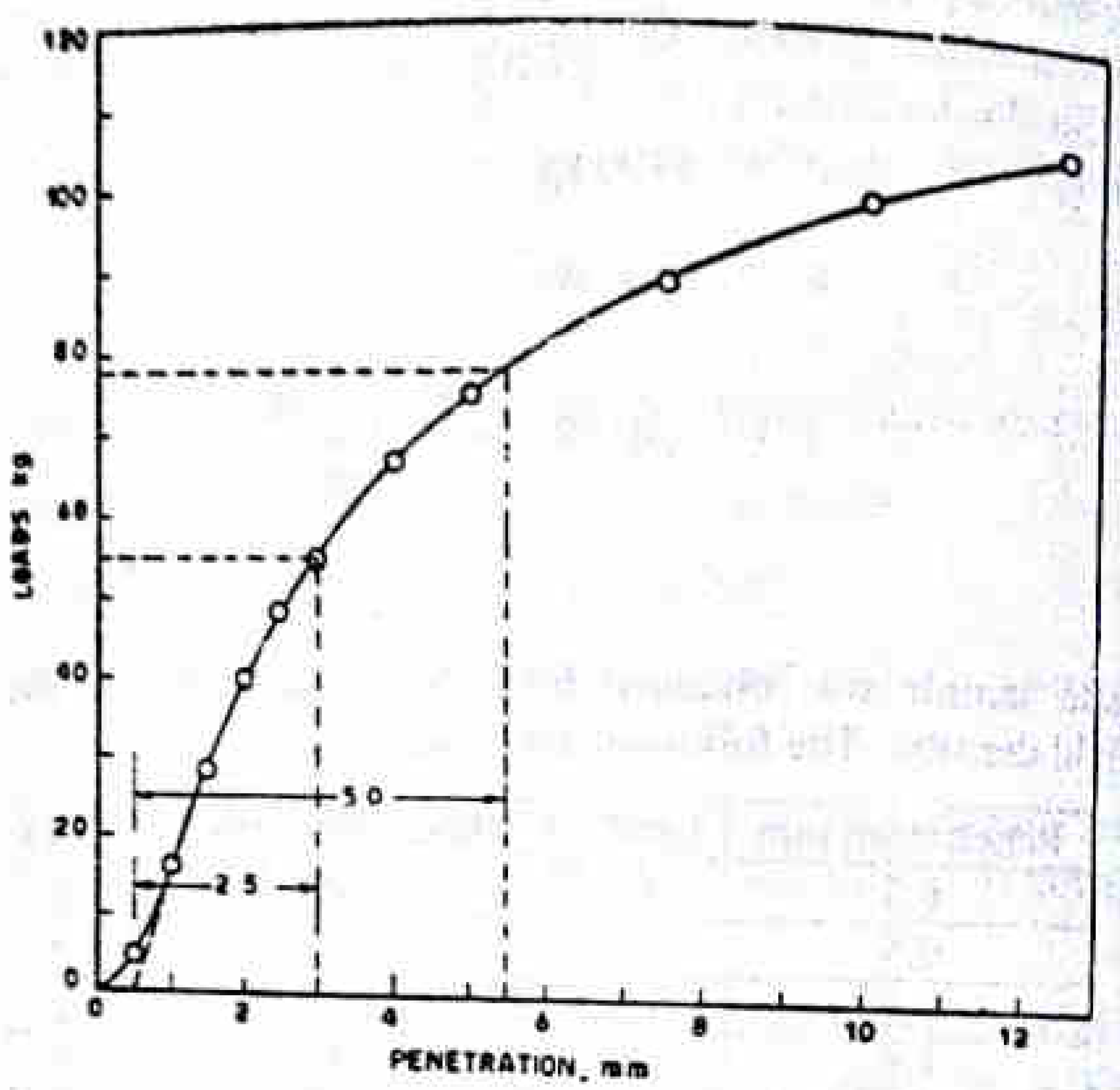


Fig. 7.12 Load-Penetration Curve (Example 7.5)

$$\text{Pressure at 5 mm penetration} = \frac{78}{19.6}\ \text{kg/cm}^2$$

$$\text{C.B.R. value of soil at 2.5 mm} = \frac{\text{Pressure on plunger @ 2.5 mm penetration for soil}}{\text{Pressure as above for standard crushed stones}} \times 100$$

$$= \frac{55}{19.6} \times \frac{100}{70} = 4.0 \text{ percent}$$

$$\text{CBR of Soil at 5 mm } \frac{78 \times 100}{19.6 \times 105} = 3.8 \text{ percent}$$

$$\text{Adopt CBR value} = 4.0 \text{ percent}$$

Calculation of Design Thickness of Different Layers

No. of vehicles for design (from Equation 7.6-c) is given by

$$A = P(1 + r)^{(n + 10)} = 1200\left[1 + \frac{8}{100}\right]^{(3+10)}$$

$$= 3260 \text{ vehicles/day}$$

Therefore Design Curve F is to be used for design as the design traffic volume is in the range 1500 to 4500 cv/day.

Using the design chart vide Fig. 7.11, the total pavement thickness over subgrade having CBR of 4 percent is obtained as 55 cm for curve F.

Thus 55 cm of pavement materials is required to cover the natural soil subgrade having 4% CBR value. Now to compute the thickness of compacted soil, the design

curve D is again used for CBR value of 7 percent. Pavement thickness of 40 cm is required above the compacted soil subgrade having CBR value of 7 percent and hence the actual thickness of this layer is 55 − 40 = 15 cm. Similarly the thickness of pavement required over poorly graded gravel of CBR 20 percent and well graded gravel of CBR 95 percent are 21 cm and 8 cm respectively.

The designed pavement section is shown in Fig. 7.13.

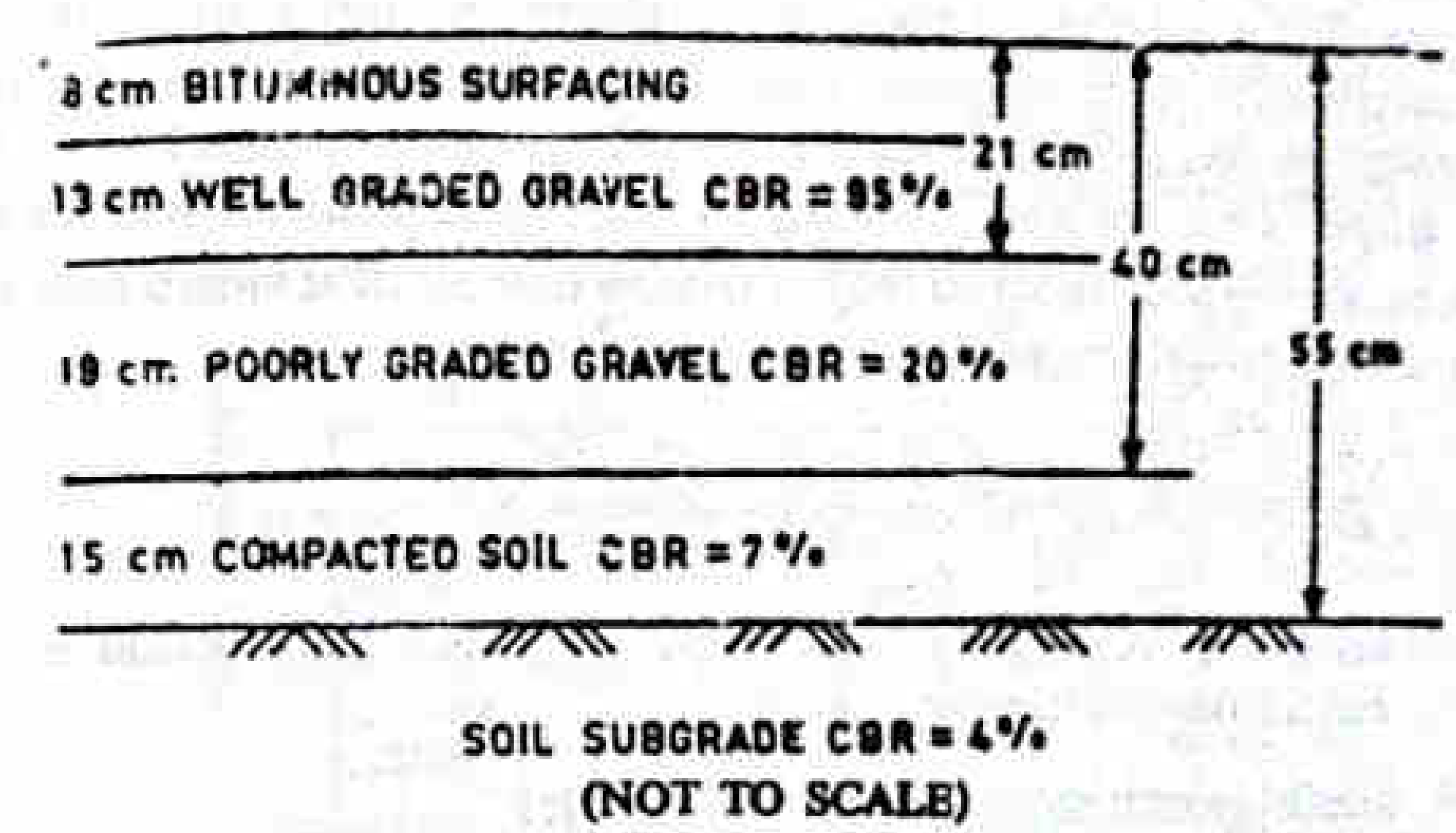


Fig. 7.13 Pavement Section by CBR method (Example 7.5)

Alternatively, if it is considered not to use poorly graded gravel as employed above, then the design section would be as shown in Fig. 7.14.

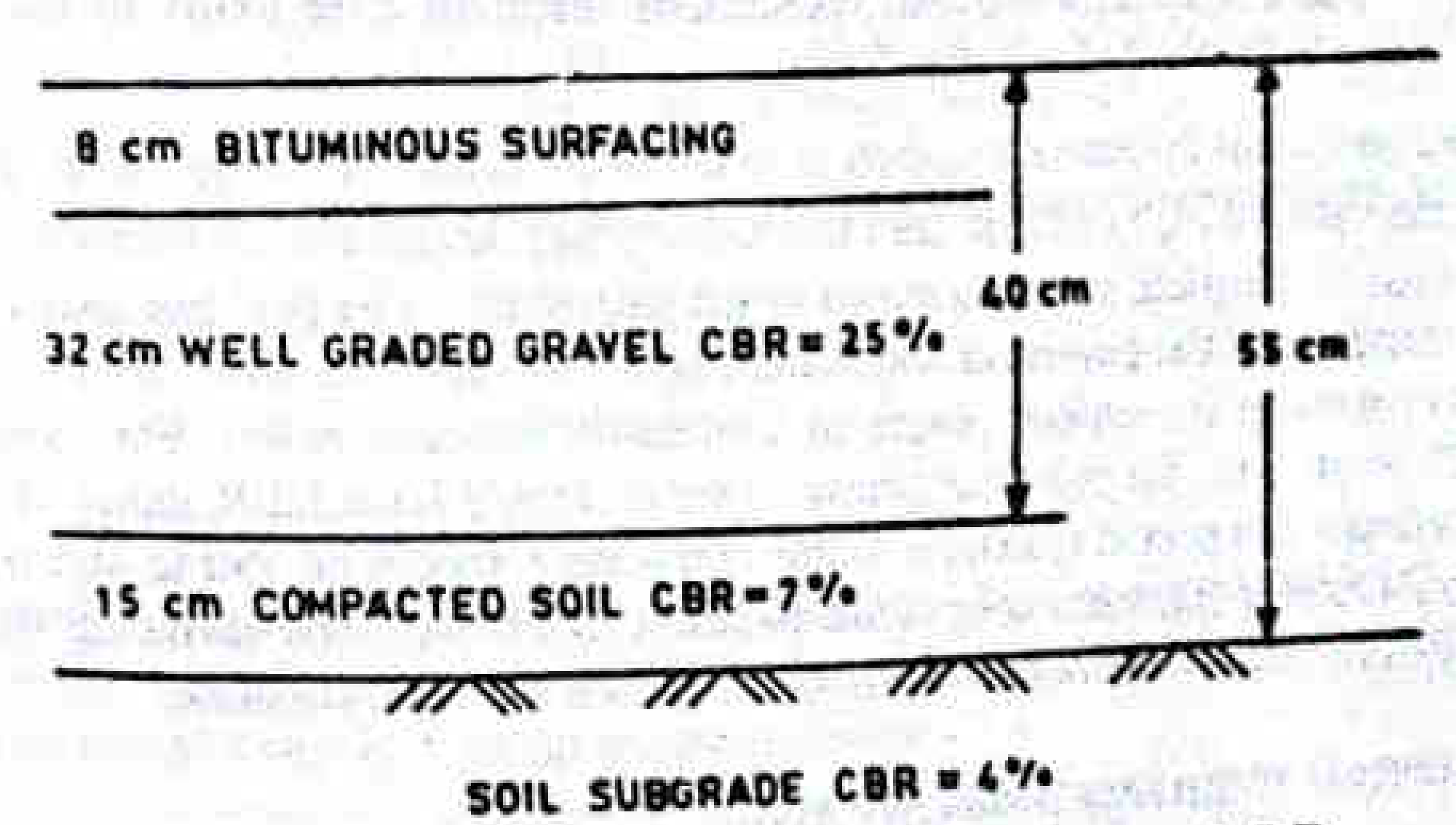


Fig. 7.14 Alternate Pavement Section (Example 7.5)

Discussion on Limitations of the Method

The CBR method suffers from one disadvantage. It may be seen that the total thickness of construction remains same i.e. 55 cm though the pavement component layers are of different materials with different CBR values. The thickness of construction over compacted soil of CBR value 7 percent is same in both cases equal to 40 cm though in one case poorly graded gravel of CBR value 20 percent is used where as in the second case, it has been replaced by well graded gravel of CBR value 95 percent

Therefore the first proposal is more economical as the thickness of well graded gravel base course is partially replaced by inferior material at the lower layer.

The CBR method of pavement design gives the total thickness requirement of the pavement above a subgrade and this thickness value would remain the same irrespective of the quality of materials used in component layers. Thus the combination of different materials should be judiciously chosen to effect durability and economy of the pavement.

CBR Method of Pavement Design by Cumulative Standard Axle Load

The Indian Roads Congress vide IRC : 37–1984 has revised the guidelines for the Design of Flexible Pavements, based on the concept of Cumulative Standard Axle Load rather than the total number of all commercial vehicles as done earlier. In the case of roads with design traffic more than 1500 commercial vehicle per day, the design traffic is defined in terms of the cumulative number of standard axle loads of 8160 kg carried during the design life of the road. The mixed commercial vehicles with different axle loads are to be converted in terms of the cumulative number of standard axle load, N_s to cater for the design, using the equation :

$$N_s = \frac{365\,A\,[(1+r)^n - 1]}{r} \times F \qquad (7.7)$$

where A = number of commercial vehicles per day when construction is completed considering the number of lanes.

r = annual growth rate of commercial vehicles

n = design life of pavement, taken as 10 to 15 years

F = vehicle damage factor, equivalent to number of standard axles per commercial vehicle on the road stretch. This is a factor converting the number of commercial vehicles of different axle loads to the number of standard axle load repetitions.

The total pavement thickness required is determined using the design chart given in Fig. 7.15, with the value of N_s in million standard axles (msa) determined as mentioned above and the CBR value of subgrade soil determined in the laboratory. The IRC has also suggested the minimum thickness of the pavement component layers of sub-base, base course and surfacing and the combinations for various ranges of cumulative standard axles. For example for the range of 20 to 30 msa, the sub-base course material should have CBR value of atleast 30% and the minimum compacted thickness of this component should be 390 to 405 mm; the base course should have a minimum compacted thickness of 250 mm and surfacing should consist of 100 to 15 mm dense bituminous macadam and 40 mm asphaltic concrete.

7.3.4 California Resistance Value Method

F. M. Hveem and *R. M. Carmany* in 1948 provided design method based on stabilometer R-value and cohesiometer C-value. The working of the stabilometer and cohesiometer are explained in article 6.4. Based on performance data, it was established by Hveem and Carmany that pavement thickness varies directly with R value and logarithm of load repetitions. It varies inversely with fifth root of C value. The expression for pavement thickness is given by the empirical equation :

$$T = \frac{K\,(TI)\,(90 - R)}{C^{1/5}} \qquad (7.8)$$

Here, T = total thickness of pavement, cm

K = numerical constant = 0.166

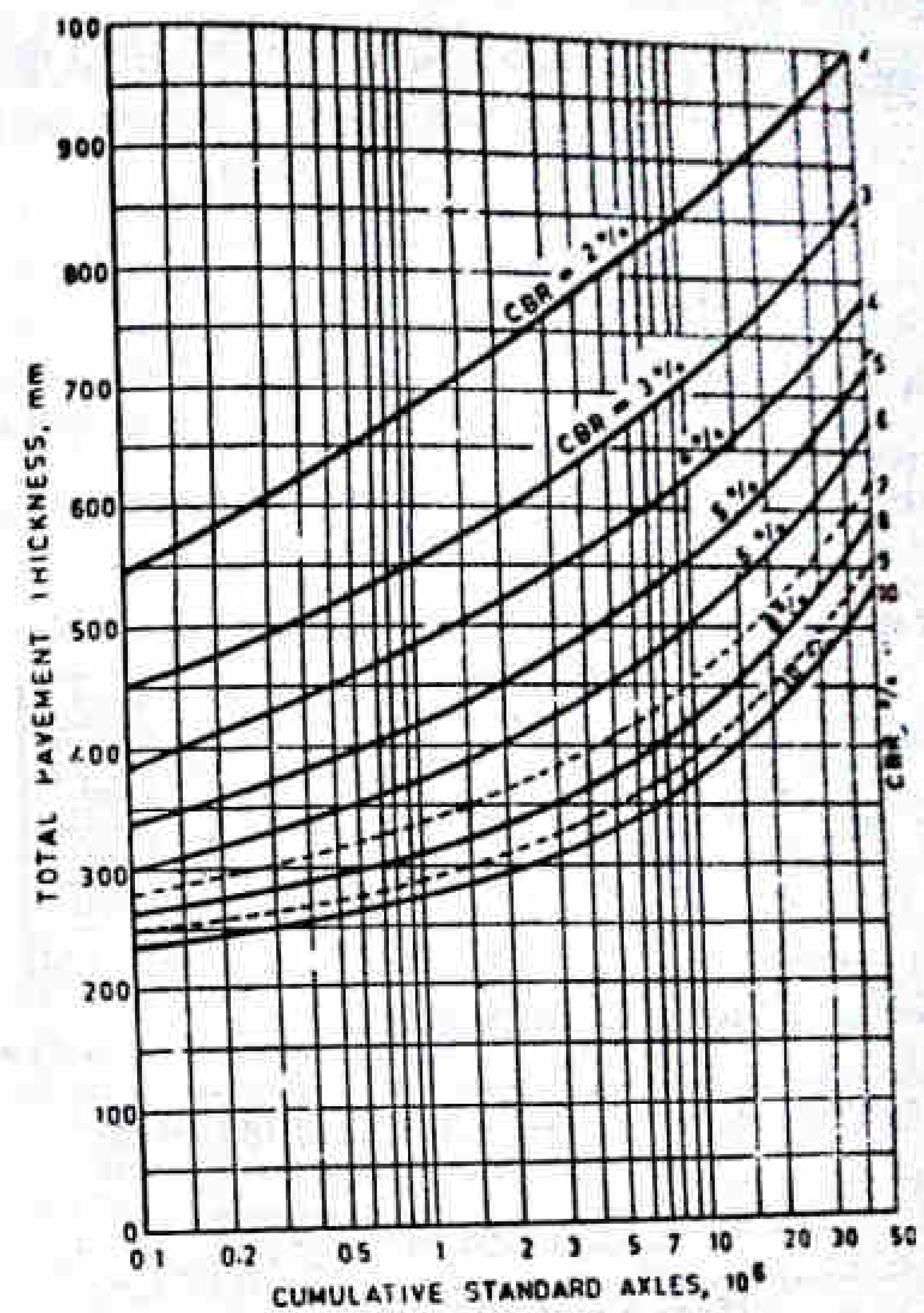


Fig. 7.15 CBR Method of Pavement Design by Cumulative Standard Axle Load

TI = traffic index = $1.35\,(EWL)^{0.11}$ (7.8a)

R = stabilometer resistance value

C = cohesiometer value

The annual value of equivalent wheel load (EWL) here is the accumulated sum of the products of the constants and the number of axle loads. The various constants for the different number of axles in a group are given below :

Number of axles	EWL constants (yearly basis)
2	330
3	1070
4	2460
5	4620
6	3040

These contents were obtained based on the State wide *loadometer survey* carried out in California during 1955-56. Hence if the annual average daily traffic volumes (AADT) data are available for different groups of axles, the yearly EWL is obtained by multiplying by the appropriate constant given above and taking the sum. Method of finding the EWL and traffic index TI has been illustrated in Example 7.6.

Example 7.6

Calculate ten-year EWL and TI values using the following AADT data

Number of axle	AADT (Two directions)
2	3500
3	344
4	295
5	80

Assume 50 percent increase in traffic in 10 year period.

Solution

The product-sum of EWL is calculated as given below :

No. of axles	AADT	EWL constant	Product
2	3500	330	1,15,5000
3	344	1070	368,080
4	295	2460	725,700
5	80	4620	369,600
		Total yearly EWL =	2,618,380

Taking the average increase for 10-year period then

$$EWL_{10} = \left(\frac{1+1.5}{2}\right) 10 \times 2,618,380$$

$$= 32,729,750$$

$$TI = 1.35\ (EWL)^{0.11}$$

$$= 1.35 \times (32,729,750)^{0.11}$$

$$= 9.057$$

In the design of flexible pavements based on California Resistance value method therefore the following data are needed :

(i) R-value of soil subgrade

(ii) TI value

(iii) Equivalent C-value of pavement materials

R value of soil subgrade is obtained from the test using stabilometer as explained in Art. 6.4 and Eq. 6.17. The computation of TI value has been explained above.

Equivalent C Value

The cohesiometer value C, is obtained for each layer of pavement material separately from tests. It is not possible to have a composite C-value for the total pavement section experimentally. However the composite or equivalent C-value of the pavement may be estimated if the thickness of each component layer and the C-value of the material of the layer is known. The method of calculating the equivalent C-value of a multilayered pavement is illustrated in Example 7.7.

While designing a pavement as the thickness of the pavement is not known, it is easier if the pavement is first assumed to consist of any one material like gravel base course with known C-value. Subsequently the individual thickness of each layer is converted in terms of gravel equivalent by using relationship :

$$\frac{t_1}{t_2} = \left(\frac{C_2}{C_1}\right)^{1/5} \qquad (7.9)$$

where, t_1 and t_2 are the thickness values of any two pavement layers and C_1 and C_2 are their corresponding cohesiometer values.

Typical C-values for some pavement materials are given below (in metric equivalents) :

Materials	C-value
Soil-cement base course	120-230
Bituminous concrete	60-62
Open graded bituminous mix	22-30
Gravel base course	15

Example 7.7

Calculate the equivalent C-value of a three layered pavement section having individual C-values as given below :

Materials	Thickness, cm	C-value
Bituminous concrete	10	60
Cement treated base	20	225
Gravel sub-base	10	15

Solution

The individual thickness of each layer is converted to their respective gravel equivalent using the following relationship :

$$\frac{t_g}{t} = \left(\frac{C}{C_g}\right)^{1/5}$$

Here, t_g = gravel thickness

t = individual thickness

C_g = cohesiometer value of gravel = 15

C = respective C-value

$$t_g = \left(\frac{60}{15}\right)^{\frac{1}{5}} \times 10 = 13.2 \text{ cm}$$

For base course, $t_g = \left(\frac{225}{15}\right)^{\frac{1}{5}} \times 20 = 34.4$ cm

For sub-base course, t_g = 10.0 cm

Therefore actual pavement thickness = 10 + 20 + 10 = 40 cm

This is equivalent to gravel thickness = 13.2 + 34.4 + 10.0 = 57.6 cm

Now,

$$\frac{t_g}{T} = \left(\frac{C}{C_g}\right)^{1/5}$$

$$C = \left(\frac{t_g}{T}\right)^5 \times C_g = \left(\frac{57.60}{40}\right)^5 \times 15 = 93$$

The equivalent C-value of the pavement section is 93,

Design Procedure

In this design method it is required to provide a pavement section which satisfies :

(i) Resistance value of sub-grade (R-value)

(ii) Expansion pressure

(iii) Exudation pressure

Laboratory tests are carried out on subgrade soil sample compacted at different moisture contents to find Hveem stabilometer R-values expansion pressure and exudation pressures. The pressure required (applied at rate of about 900 kg per minute) to force out water from a compacted subgrade soil sample is known as exudation pressure and this depends on soil type and the moisture content. As the compacting moisture content of the soil is increased, the R value, exudation and expansion pressure decreases.

In pavement design problems, first the pavement thickness required may be calculated assuming it to consist of a single layer material of known C-value such as gravel or water bound macadam (WBM) base course. Subsequently the thickness of the other component layers are chosen as per the traffic and climatic requirements and the equivalent base course layer thickness to be replaced by these pavement layers are calculated based on their C-values using Equation 7.9.

Design steps

(i) The pavement thickness values required as per the R-values of subgrade soil at different moisture contents are calculated (say, T_{r1}, T_{r2} ...) using Equation 7.8. Here the pavement may first be assumed to consist of single base course layer of known C-value, C_g.

(ii) The pavement thickness values required to counteract the subgrade expansion pressure are found by dividing the expansion pressure by the average density of the pavement which may be assumed as about 2.1 g/cm^2. The pavement thickness value (say Te_1, Te_2 ...) as per expansion pressures at different moisture contents are calculated.

(iii) The pavement thickness fulfilling both R-value and expansion pressure is found by plotting T_r values against the corresponding T_e values from (i) and (ii) above, to the same scale and by drawing a 45° line so that $T_e = T_e$.

(iv) The exudation pressure of subgrade soil found at various compacting moisture contents are plotted against the pavement thickness found from (i) above based on the corresponding R values. The pavement thickness corresponding to an exudation pressure of 28 kg/cm^2 is obtained from this graph, say Td.

(v) The pavement thickness as per California design method is the higher of the values determined in (iii) and (iv) above.

(vi) The thickness of other pavement layers are decided and the equivalent values of base course thickness replaced are calculated using Eq. 7.9 with the known Cohesiometer values of the materials.

The California R-values method of pavement design is a purely empirical method and therefore the test procedure and the specifications should be strictly followed. Nomograms are also available to simplify the design calculations.

The pavement design method is illustrated in Example 7.8.

Example 7.8

Design a flexible-pavement consisting of water bound macadam (WBM) base course and bituminous concrete surface course of thickness 7.5 cm by California R-value (Stabilometer) method using the following data :

Moisture content %	R value	Pressure, kg/cm^2	
		Expansion	Exudation
15	56	0.135	46.5
18	44	0.099	41.5
21	25	0.055	30.5
24	14	0.034	21.5

(i) Test results on subgrade soil

(ii) C-value of WBM base course = 15

(iii) C-value of bituminous concrete surface course = 62

(iv) Traffic index = 9.5

Solution

First find the pavement thickness required using WBM base material only (C value = 15).

(i) Thickness by R-values

$$T = \frac{K(TI)(90 - R)}{C^{1/5}} = \frac{0.166 \times 9.5 \times (90 - 56)}{15^{1/5}} = 31.2 \text{ cm}$$

Similarly for R-values of 44, 25 and 14 the pavement thickness values are 42.2, 59.6 and 69.7 cm respectively.

(ii) Thickness by Expansion Pressure

Assuming average pavement density as 2.1 g/cm^3 or 0.0021 kg/cm^3, the pavement thickness needed for counteracting expansion pressure of 0.135 kg/cm^2 = $\frac{0.135}{0.0021}$ = 64.3 cm. Similarly, pavement thickness for expansion pressures of 0.099, 0.055 and 0.034 kg/cm^2 are 47.1, 26.2 and 16.2 cm respectively.

The pavement thickness values obtained by R-values and expansion pressure values at various subgrade moisture contents along with the corresponding exudation pressure are given in the Table below :

Moisture content %	R value	Pressure, kg/cm²	
		Expansion	Exudation
15	56	0.135	36.5
18	44	0.099	26.5
21	25	0.055	18.0
24	14	0.034	15.0

The pavement thickness values given in this table are plotted in Fig. 7.16 and the equal thickness value by the two methods is obtained by drawing a 45° line. The thickness obtained in this case is $T_r = T_e = 44.5$ cm.

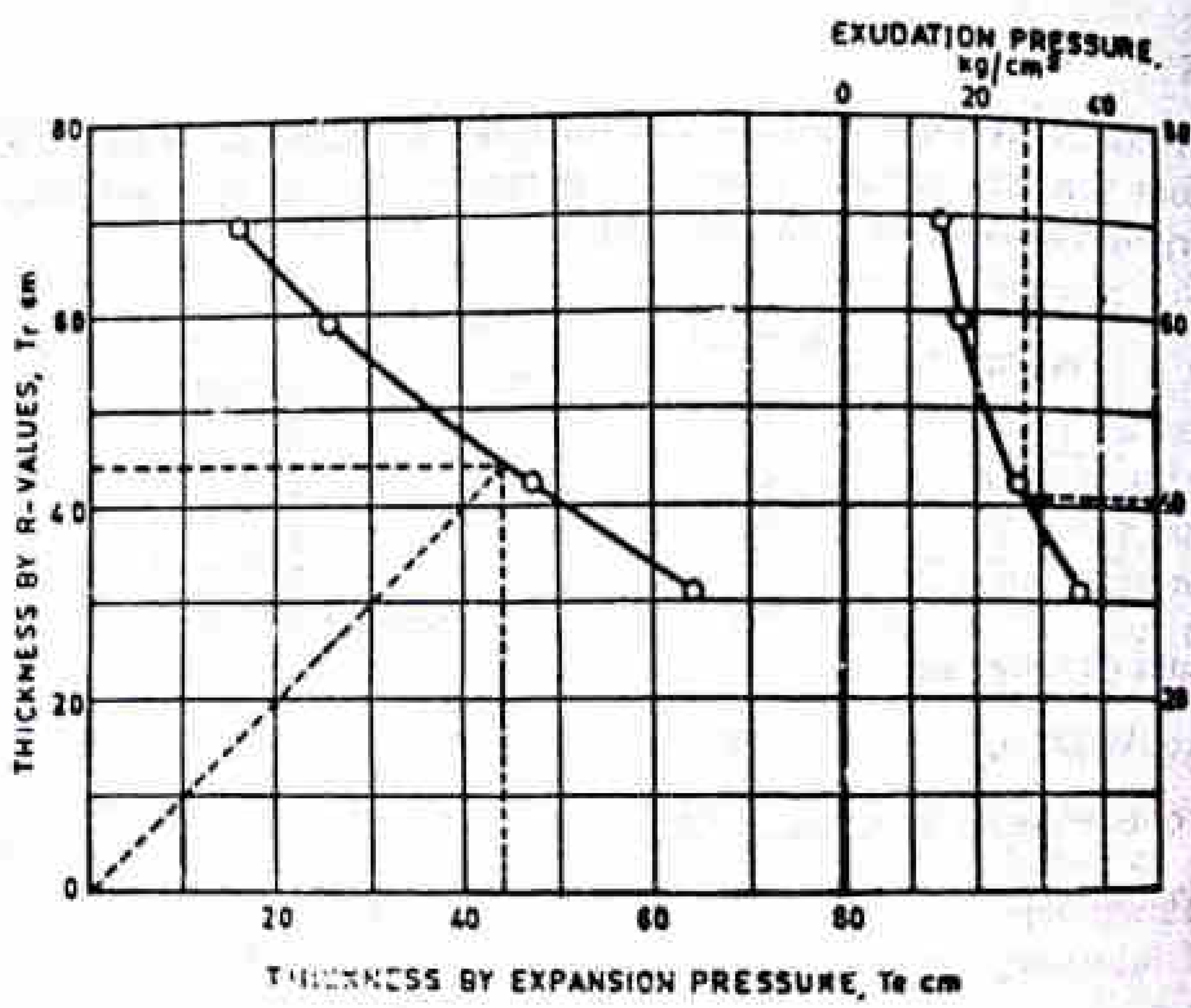


Fig. 7.16 Design of Pavement by R-value Method (Example 7.8)

(iii) Figure 7.16 also shows the plot between the exudation pressure values at the four moisture contents and the corresponding values of pavement thickness calculated based on R-values at these moisture contents. The pavement thickness corresponding to an exudation pressure of 28 kg/cm² obtained is 40.5 cm from this plot.

(iv) Therefore the design thickness of single layer WBM pavement is higher of the two values (44.5) and (40.5 cm) and is equal to 44.5 cm.

(v) Bituminous concrete surface course thickness $t_c = 7.5$ cm.

C-value of bituminous concrete $C_c = 62$

C-value of WBM base course $C_b = 15$

Equivalent thickness of 7.5 cm bituminous concrete in terms of WBM base course t_b is given by the relation

$$\frac{t_b}{t_c} = \left(\frac{C_c}{C_b}\right)^{1/5} \quad \text{i.e., } \frac{t_b}{7.5} = \left(\frac{62}{15}\right)^{1/5}$$

$$t_b = 7.5\left(\frac{62}{15}\right)^{1/5} = 10.0 \text{ cm}$$

WBM base course thickness required = 44.5 – 10.0 = 34.5 cm

Therefore the pavement section consists of 34.5 cm of WBM base course and 7.5 cm of bituminous concrete. The designed pavement section is shown in Fig. 7.17.

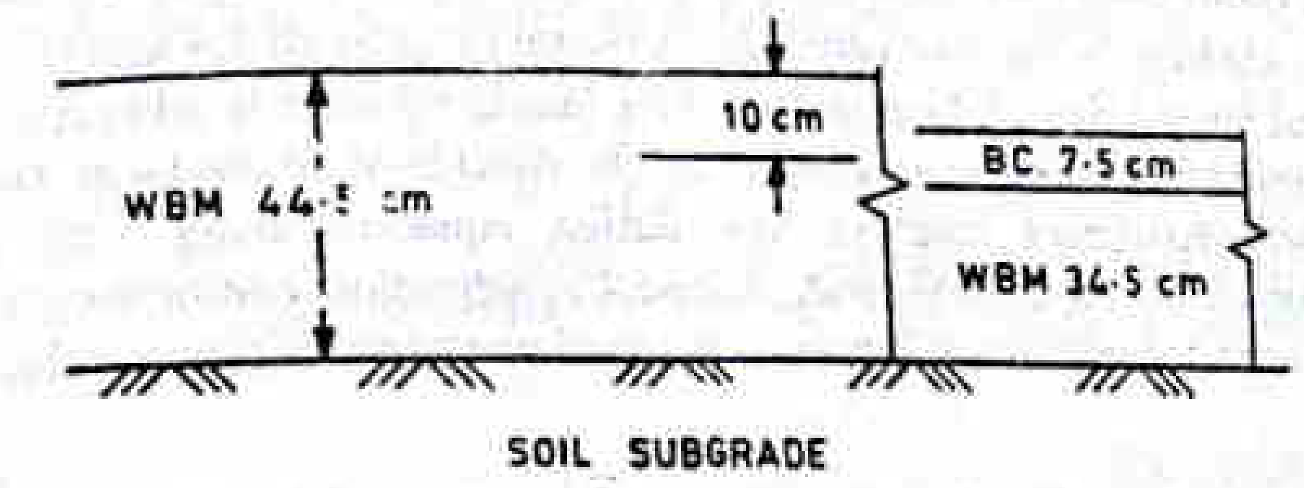


Fig. 7.17 Pavement Section (Example 7.8)

7.3.5 Triaxial Method

L. A. Palmer and *E. S. Barber* in 1910 proposed the design method based on Boussinesq's displacement equation for homogeneous elastic single layer:

$$\Delta = \frac{3pa^2}{2E(a^2+z^2)^{1/2}} \tag{7.10}$$

Here $p = P/\pi a^2$

$$\therefore \Delta = \frac{3P}{2\pi E(a^2+z^2)^{1/2}}$$

$$(a^2+z^2)^{1/2} = \frac{3P}{2\pi E\Delta}$$

$$(a^2+z^2) = \left(\frac{3P}{2\pi E\Delta}\right)^2$$

$$z = \sqrt{\left(\frac{3P}{2\pi E\Delta}\right)^2 - a^2}$$

Assuming that the pavement is incompressible, z becomes T, the thickness of pavement.

$$T = \sqrt{\left(\frac{3P}{2\pi E_s\Delta}\right)^2 - a^2} \tag{7.11}$$

Here T = pavement thickness, cm

P = wheel load, kg

E_s = modulus of elasticity of subgrade from triaxial test results, kg/cm².

a = radius of contact area, cm

Δ = design deflection (0.25 cm)

In the above analysis the pavement and the subgrade are assumed to have the same E-value.

Use of Triaxial Test

The triaxial compression test as explained in article 6.1.8 is used in determining the values of elastic modulii for various materials. A lateral pressure of 1.4 kg/cm^2 is applied in the test to find the E value of the material. This lateral pressure is arbitrarily assumed as the lateral confinement in pavement layers by the *Kansas State Highway Department* of USA. This department employs this design equation along with empirical modifications for : (i) traffic coefficient, X and (ii) saturation coefficient, Y. These coefficients are used as multiplying factors to the total pavement thickness value which is thus modified.

The pavement thickness T_s consisting of material with modulus E_s is given by the equation :

$$T_s = \sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2} \qquad (7.12)$$

The recommended values of coefficients X and Y based on ADT of design traffic and rainfall are given below :

Traffic coefficient (X)	ADT (number)
1/2	40 – 400
2/3	401 – 800
5/6	801 – 1200
1	1201 – 1800
7/6	1801 – 2700
8/6	2701 – 4000
9/6	4001 – 6000
10/6	6001 – 9000
11/6	9001 – 13,500
12/6	13501 – 20,000

Rainfall coefficient (Y)	Average annual rainfall, cm
0.5	38 – 50
0.6	51 – 64
0.7	65 – 76
0.8	77 – 90
0.9	91 – 100
1.0	101 – 127

If pavement and subgrade are considered as a two layer system, a *Stiffness factor* has to be introduced to take into account the different values of modulus of elasticity of the two layers. The pavement thickness is then modified using the stiffness factor equal to $(E_s/E_p)^{1/3}$ where E_s and E_p are values of modulus of elasticity of the subgrade and pavement, respectively. Thus the thickness of pavement, T_p is calculated from the relation :

$$T_p = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_p}\right)^{1/3} \qquad (7.13)$$

The thickness design equation 7.11 is based on elastic theory. However the modified equations taking into account the traffic and rainfall coefficients and stiffness factor (Equations 7.12 and 7.13) are empirical modifications. The relation between pavement layers of thickness t_1 and t_2 of elastic modulus E_1 and E_2 is given by :

$$\frac{t_1}{t_2} = \left(\frac{E_2}{E_1}\right)^{1/3}$$

Thus the Kansas Highway Department design method may be categorized as semi-theoretical or semi-empirical method, using triaxial test results.

Example 7.9

Design the pavement section by triaxial test method using the following data :

Wheel load = 4100 kg

Radius of contact area = 15 cm

Traffic coefficient, X = 1.5

Rainfall coefficient Y = 0.9

Design deflection Δ = 0.25 cm

E-value of subgrade soil E_s = 100 kg/cm^2

E-value of base course material E_b = 400 kg/cm^2

E-value of 7.5 cm thick bituminous concrete surface course = 1000 kg/cm^2

Solution

Assuming the pavement to consist of single layer of base course material only; the pavement thickness is given by :

$$T_b = \left\{\sqrt{\left(\frac{3P\,XY}{2\pi E_s \Delta}\right)^2 - a^2}\right\}\left(\frac{E_s}{E_b}\right)^{1/3}$$

$$= \left\{\sqrt{\left(\frac{3\times 4100\times 1.5\times 0.9}{2\pi\times 100\times 0.25}\right)^2 - 15^2}\right\}\left(\frac{100}{400}\right)^{\frac{1}{3}} = 104.64\times 0.63 = 65.9 \text{ cm}$$

Let 7.5 cm bituminous concrete surface with E_c = 1000 kg/cm^2 be equivalent to the thickness t_b of base course. The equivalent replacement t_b is obtained from the relation :

$$\frac{t_b}{t_c} = \left(\frac{E_c}{E_b}\right)^{1/3} \text{ i.e., } \frac{t_b}{7.5} = \left(\frac{1000}{400}\right)^{1/3}$$

$$t_b = 7.5\times\left(\frac{1000}{400}\right)^{1/3} = 10.2 \text{ cm}$$

Therefore the required base course thickness = 65.9 – 10.2 = 55.7 cm

The pavement section consists of 55.7 cm thick WBM base course and 7.5 cm thick bituminous concrete surface course. See Fig. 7.18.

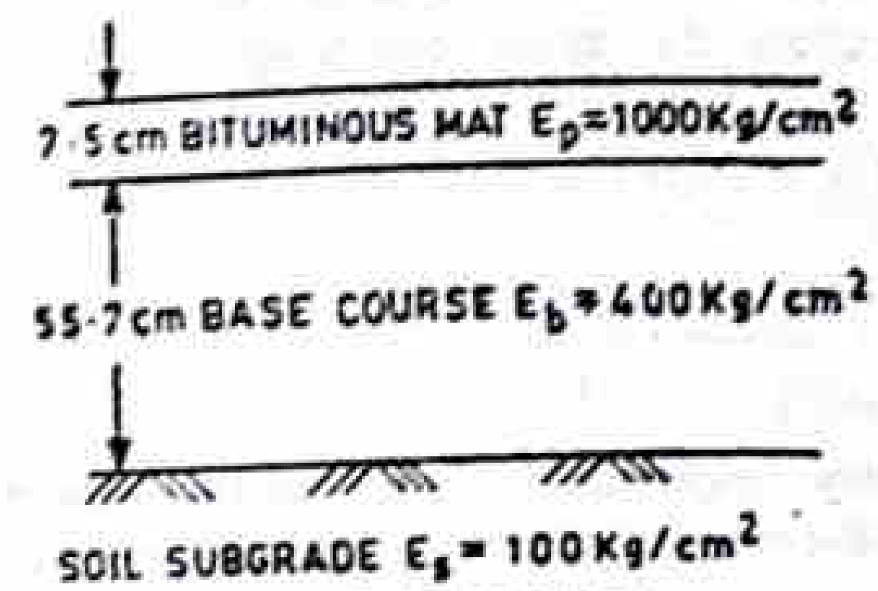


Fig. 7.18 **Pavement Section with Base Course**

7.3.6 McLeod Method

Norman W. McLeod through Canadian Department of Transport conducted extensive plate bearing tests on airfield and highway pavements and developed a design method. The repetitive plate bearing test procedure was employed using various sizes of plates.

From the plate load tests an empirical design equation was recommended :

$$T = K \log_{10} \frac{P}{S} \tag{7.14}$$

Here T = required thickness of gravel base, cm

P = gross wheel load, kg

= total subgrade support, kg (for the same contact area, deflection and number of repetitions of load P)

K = base course constant.

It is found that the base course constant K depends on the loaded area. Figure 7.19 shows the relationship between the plate diameter and base course constant. Thus the granular base course thickness requirement T may be calculated from Eq. 7.14 for a given wheel load P if the subgrade support is known from plate bearing test data. The subgrade support S for the design of highway pavement is calculated from the support measured or calculated for 30 cm diameter plate at 0.5 cm deflection and ten repetitions. Figure 7.20 is used for finding the ratio of unit subgrade support for the design wheel load diameter to that on 30 cm diameter plate at 0.5 cm deflection. The design unit subgrade support is obtained by multiplying the contact pressure of the design load by the above ratio. The total subgrade support S is calculated by multiplying the unit support by the contact area. The design method is illustrated in Example 7.10.

Example 7.10

Design a highway pavement for a wheel load of 4100 kg with a tyre pressure of 5 kg/cm² by McLeod method. The plate bearing test carried out on subgrade soil using 30 cm diameter plate yielded a pressure of 2.5 kg/cm² after 10 repetitions of load at 0.5 cm deflection.

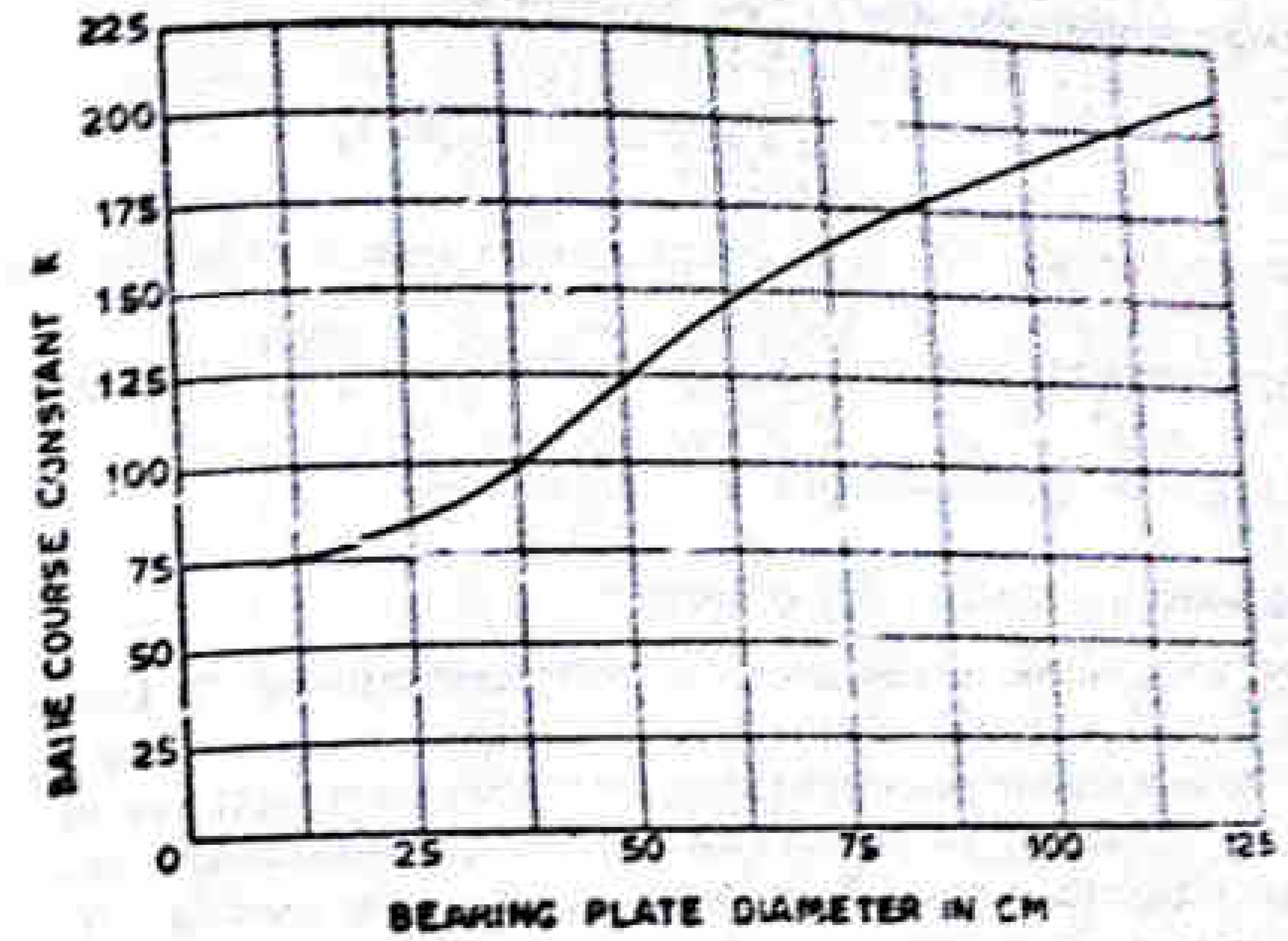


Fig. 7.19 **Relation between Plate Diameter and Base Course Constant**

Solution

$$\text{Radius of contact, } a = \sqrt{\frac{P}{p\pi}} = \sqrt{\frac{4100}{5\pi}} = 16.1 \text{ cm}$$

$$\text{Perimeter over area ratio, } \frac{P}{A} = \frac{2}{a} = \frac{2}{16.1} = 0.124$$

Using Fig. 7.20 the ratio of unit subgrade support on 32.2 cm diameter plate at 0.5 cm deflection is 0.95.

Therefore, unit support at 0.5 cm deflection

$$= 0.95 \times 2.5 = 2.44 \text{ kg/cm}^2$$

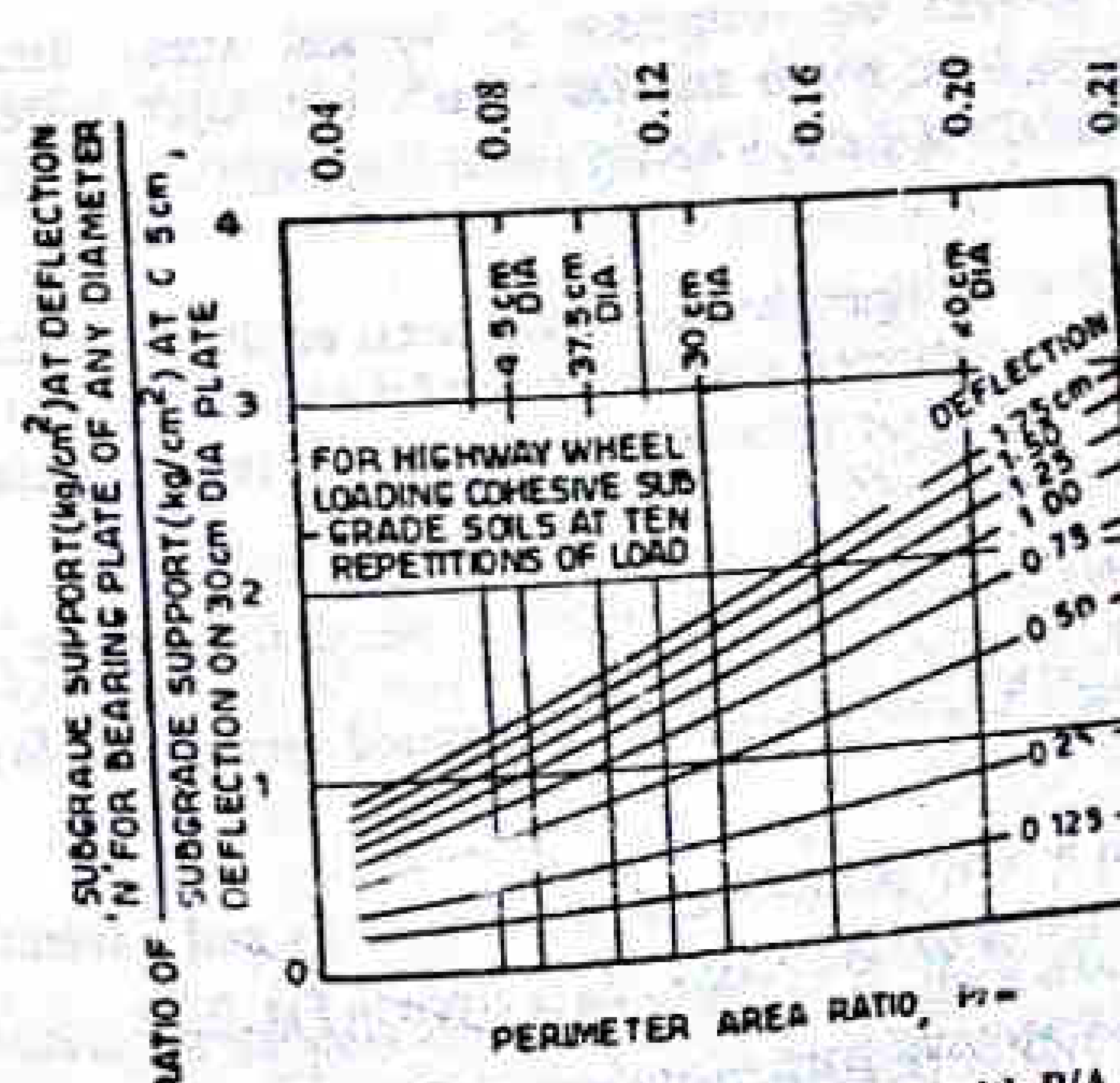


Fig. 7.20 **Relationship of Subgrade Support with P/A ratio**

Design subgrade support on 32.2 cm diameter plate,

$$S = 2.44\,\pi\,\frac{32.2^2}{4} = 2100 \text{ kg}$$

Base course constant for 32.2 cm diameter plate is obtained from Fig. 7.19 as 90.

$$\text{Granular pavement thickness, } T = K \log_{10}\frac{P}{S} = 90 \log_{10}\frac{4100}{2100} = 26 \text{ cm}$$

Provide 5 cm of bituminous surfacing out of this thickness.

7.3.7 Burmister's (Layered System) Method

Donald M. Burmister developed the layered system analysis. As known the flexible pavement sections are composed of layers and the elastic modulus of the top layer is the highest. The total mass of pavement and subgrade does not possess a constant E value as assumed by Boussinesq in his analysis. However, Boussineq's analysis can be considered as a special case of Burmister's layered system analysis. If layers of soil subgrade, sub-base course and base course are assigned elastic modulii of E_s, E_{sb}, E_b then as per Boussineq's analysis, it is considered $E_s = E_{sb} = E_b$ whereas in layered analysis, it is taken that $E_b > E_{sb} > E_s$. The effectiveness of the reinforcing action of the pavement layers is logically utilised in Burmister's approach. Following are the assumptions made in this approach :

(i) the materials, in the pavement layers are isotropic, homogeneous and elastic. The pavement forms a stiffer reinforcing layer having modulus of elasticity higher than that of the underlying subgrade in the two layer system.

(ii) the surface layer is infinite in horizontal direction and finite in vertical direction; the underlying layer in two layered system is considered infinite in both directions.

(iii) the layers are in continuous contact; the top layer is free of shearing and normal stresses outside the loaded area.

Figure 7.21 provides the comparison of vertical stress distribution between Boussinesq's single layer system and Burmister's two layer system, assuming the pavement to consist of a single layer having elastic modulus E_p lying over subgrade with elastic modulus Es.

It is observed from this figure that the vertical stress on the subgrade is reduced from 70 to 30 percent by introducing a pavement layer of thickness equal to the radius of the load or h = a, having elastic modulus 10 times higher than the elastic modulus of subgrade soil i.e., for $E_p/E_s = 10$.

The Burmister's approach therefore utilises the reinforcing action of the pavement layer.

The deflection factor F_2 is introduced in two layered system which is dependent on E_s/E_p and h/a.

The relationship between two layer deflection factor F_2 and pavement thickness in terms of radius a of loaded area and ratios E_s/E_p is given in Fig. 7.22.

The displacement equations given by Burmister (Equations 7.4 and 7.5) are written here :

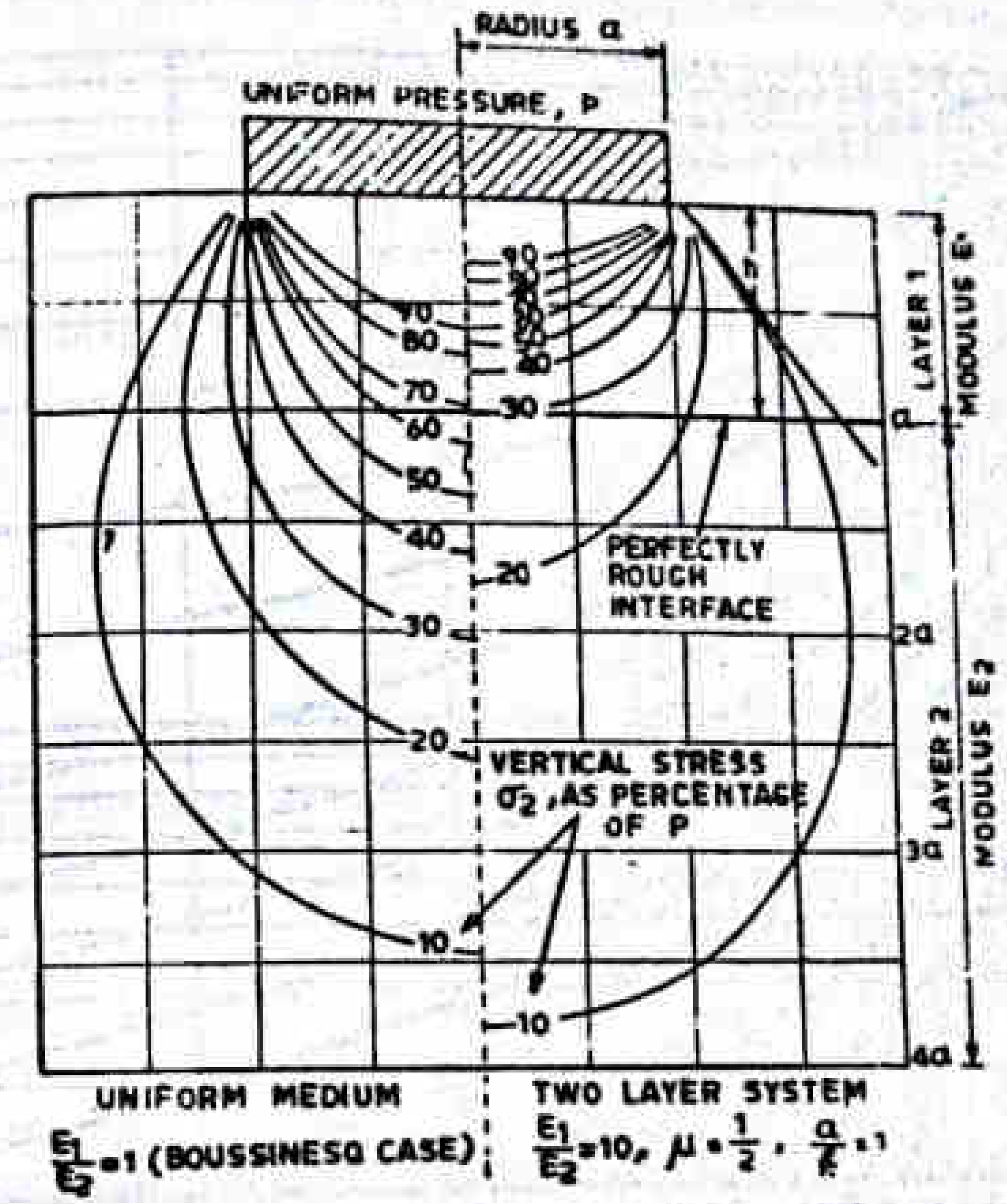


Fig. 7.21 Comparison of Vertical Stress Distribution by Boussinesq and Burmister Approaches

For flexible plate, $\Delta = 1.5\,\frac{pa}{E_s}.F_2$

For rigid plate, $\Delta = 1.18\,\frac{pa}{E_p}.F_2$

For single layer, h = 0, and $E_s/E_p = 1$ therefore $F_2 = 1$ and these equations reduce to Boussinesq's settlement equation (Eq. 7.2 and 7.3). See Figure 7.22. In the derivations of displacement equations the Poisson's ratio μ is taken as 0.5 both for subgrade and pavement material,

i.e. $\mu_s = \mu_p = 0.5$

The above analysis is adopted by U.S. Navy Department for design of *air field* pavements. It is considered that the layered system analysis can also be applied for design of highway pavements. Following assumptions can be suitably made. The plate diameter for load tests may be taken as 30 cm and design deflection may be taken as 0.5 or 0.25 cm. The design method using Burmister's two-layer theory is illustrated in Example 7.11.

Example 7.11

The plate bearing tests were conducted with 30 cm plate diameter on soil subgrade and over 15 cm base course. The pressure yielded at 0.5 cm deflection are 1.25 kg/cm^2 and 4.0 kg/cm^2, respectively.

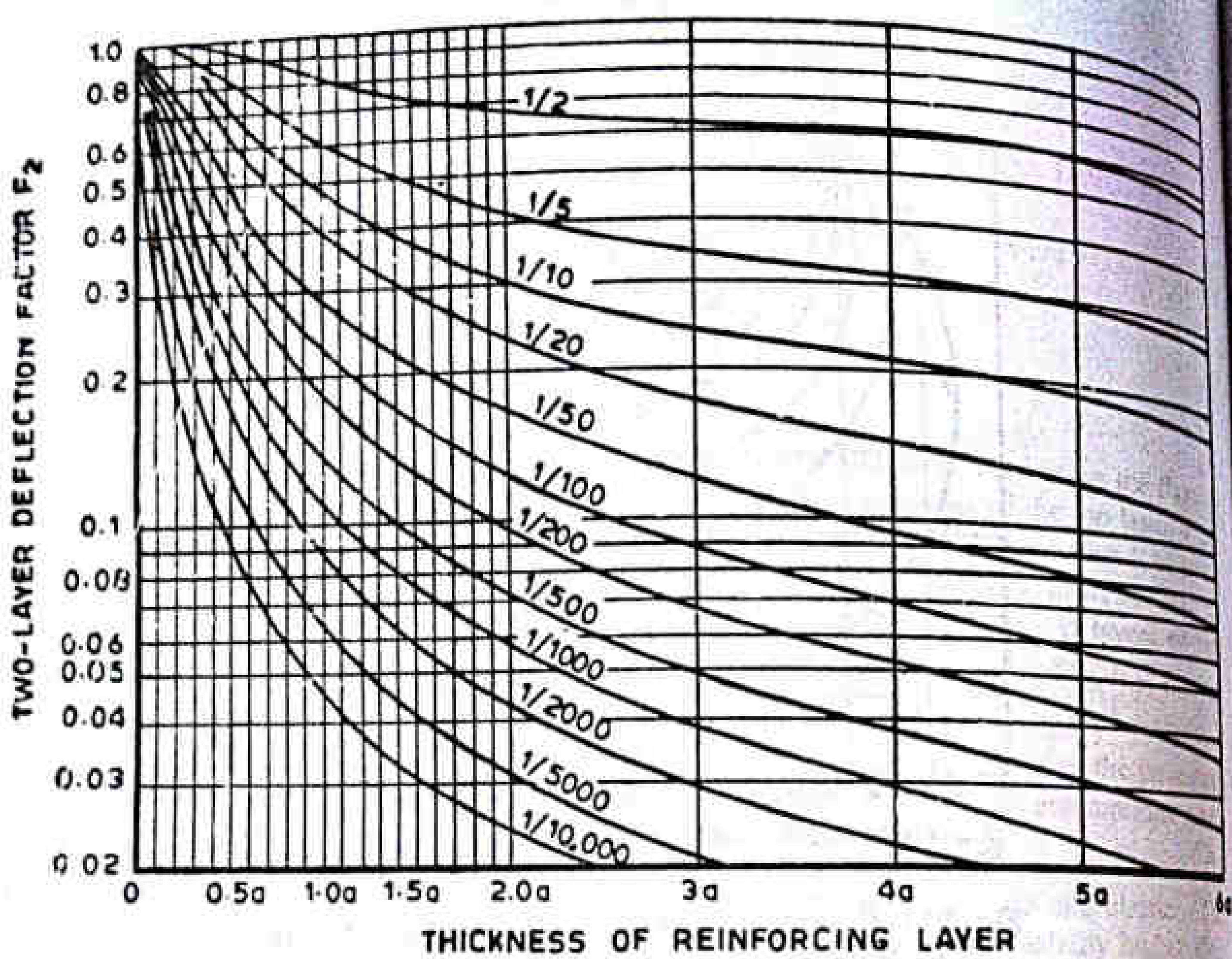


Fig. 7.22 Relationship of F2 and h in a Two-lane System (Burmister's Method)

Design the pavement section for 4100 kg wheel load with tyre pressure of 5 kg/cm^2 for an allowable deflection of 0.5 cm using Burmister's approach.

Solution

(i) Calculate elastic modulus, E_s for soil subgrade or single layer and rigid circular plate,

$$\Delta = 1.18\,\frac{pa}{E_s}\,F_2;\ \text{i.e. } 0.5 = \frac{1.18 \times 1.25 \times 15 \times 1}{E_s}$$

$$F = 1 \text{ (for single layer)}$$

$$E_s = 1.18\,\frac{1.25 \times 15}{0.5} = 44.2 \text{ kg/cm}^2$$

(ii) Calculate elastic modulus ratio of subgrade to pavement E_s/E_p

$$\Delta = \frac{1.18\,pa}{E_s} \times F_2;\ \text{i.e. } 0.5 = \frac{1.18 \times 4 \times 15}{44.2} \times F_2$$

or,

$$F = \frac{0.5 \times 44.2}{1.18 \times 4 \times 15} = 0.312$$

Use Fig. 7.22 and read value of E_s/E_p against $F_2 = 0.312$ and $(h/a) = (15/15) = 1.0$

$$\frac{E_s}{E_p} = \frac{1}{40} \text{ (by interpolation between 1/20 and 1/50)}$$

(iii) Design of flexible pavement for load P = 4100 kg and tyre pressure of 5 kg/cm^2

$$p = 5 \text{ kg/cm}^2$$

$$a = \sqrt{\frac{P}{\pi p}} = \sqrt{\frac{4100}{\pi \times 5}} = 16.1 \text{ cm}$$

Deflection for flexible plate (wheel load) is given by :

$$\Delta = \frac{1.5\,pa}{E_s} \times F_2;\ \text{i.e. } 0.5 = \frac{1.5 \times 5 \times 16.1 \times F_2}{44.2}$$

$$F_2 = \frac{0.5 \times 44.2}{1.5 \times 5 \times 16.1} = 0.183$$

For $F_2 = 0.183$ and $(E_s/E_p) = (1/40)$ using Fig. 7.20, h/a = 2.1

Therefore pavement thickness 'h' is given by :

$$h = 2.1\,a = 2.1 \times 16 = 33.6 \text{ cm}$$

For airfield pavement design by U. S. Navy method modifications of theoretically calculated thickness have been suggested. It has been recommended that nine trial sections be constructed three each on fill, cut and level areas. In each typical locality, three pavement thickness values equal to 2/3 h, h and 1.5 h are adopted and the actual pavement thickness required for the critical deflection is found. Similar approach is possible in highway pavement design also after calculating the thickness requirement by the elastic layer theory.

7.4 DESIGN OF RIGID PAVEMENTS

7.4.1 General Design Considerations

Cement concrete pavements represent the group of rigid pavements. Here the load carrying capacity is mainly due to the rigidity and high modulus of elasticity of the slab itself i.e., *slab action*. *H. M. Westergaard* is considered the pioneer in providing the rational treatment to the problem of rigid pavement analysis.

Westergaard considered the rigid pavement slab as a thin elastic plate resting on soil subgrade, which is assumed as a dense liquid. Here it is assumed that the upward reaction is proportional to the deflection, i.e., $p = K\,\Delta$, where the constant K is defined as modulus of subgrade reaction. The unit of K is kg/cm^2 per cm deflection i.e., kg/cm^3.

Westergaard's Modulus of Subgrade Reaction

The modulus of subgrade reaction, K is proportional to the displacement. The displacement level Δ is taken as 0.125 cm in calculating K as explained in Art. 6.1.8. If p is the pressure sustained in kg/cm^2 by the rigid plate of diameter 75 cm at a deflection $\Delta = 0.125$ cm, the modulus of subgrade reaction K is given by :

$$K = \frac{p}{\Delta} = \frac{p}{0.125} \text{ kg/cm}^3$$

Relative Stiffness of Slab to Subgrade

A certain degree of resistance to slab deflection is offered by the subgrade. This is dependent upon the stiffness or pressure-deformation properties of the subgrade material. The tendency of the slab to deflect is dependent upon its properties of flexural strength.

The resultant deflection of the slab which is also the deformation of subgrade is a direct measure of the magnitude of subgrade pressure. The pressure deformation characteristics of rigid pavement is thus a function of relative stiffness of slab to that of subgrade.

Westergaard defined this term as the *Radius of relative stiffness*

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{\frac{1}{4}} \quad (7.15)$$

Here l = radius of relative stiffness, cm

E = modulus of elasticity of cement concrete kg/cm^2

μ = Poisson's ratio for concrete = 0.15

h = slab thickness, cm

K = subgrade modulus or modulus of subgrade reaction, kg/cm^3

Example 7.12

Compute the radius of relative stiffness of 15 cm thick cement concrete slab from the following data :

Modulus of elasticity of cement concrete = 2,10,000 kg/cm^2

Poisson's ratio for concrete = 0.13

Modulus of subgrade reaction, K = (i) 3.0 kg/cm^3 (ii) 7.5 kg/cm^3

Solution

(i) For K = 3.0

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{\frac{1}{4}} = \left[\frac{210000 \times 15^3}{12 \times 3(1-0.15^2)}\right]^{\frac{1}{4}} = 67.0 \text{ cm}$$

(i) For K = 7.5

$$t = \left[\frac{210000 \times 15^3}{12 \times 7.5(1-0.15^2)}\right]^{\frac{1}{4}} = 53.3 \text{ cm}$$

This indicates that the influence of modulus of subgrade reaction on the slab is relatively small.

The stresses acting on a rigid pavement are ;

(i) wheel load stresses and

(ii) temperature stresses.

Critical Load Position

Since the pavement slab has finite length and width, either the character or intensity of maximum stress induced by the application of a given traffic load is dependent on the location of the load on the pavement surface.

There are three typical locations namely the interior, edge and corner, where differing conditions of slab continuity exist. These are termed as *critical load positions*.

Interior Loading : When load is applied in the interior of the slab surface at any place remote from all the edges.

Edge Loading : When load is applied on an edge of the slab at any place remote from a corner.

Corner Loading : When the centre of load application is located on the bisector of the corner angle formed by two intersecting edges of the slab, and the loaded area is at the corner touching the two corner edges.

Equivalent Radius of Resisting Section

Considering the case of interior loading, the maximum bending moment occurs at the loaded area and acts radially in all directions. With the load concentrated on a small area of the pavement, the question arises as to what sectional area of the pavement is effective in resisting the bending moment. According to Westergaard, the equivalent radius of resisting section is approximated, in terms of radius of load distribution and slab thickness,

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h \quad (7.16)$$

Here, b = equivalent radius of resisting section, cm when a is less than 1.724 h

a = radius of wheel load distribution, cm

h = slab thickness, cm

when a is greater than 1.724 h, the value of b = a

Example 7.13

Compute the equivalent radius of resisting section of 20 cm slab, given that the radius of contact area wheel load is 15 cm.

Solution

$$h = 20 \text{ cm}, a = 15$$

$$\frac{a}{h} = \frac{15}{20} = 0.75, < 1.724$$

Therefore

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h$$

$$= \sqrt{1.6 \times 15^2 + 20^2} - 0.675 \times 20 = 14.07 \text{ cm}$$

Maximum stress produced by a wheel load at corner does not exist around the load, but it occurs at some distance X along the corner bisector. Thus is given by the relation :

$$X = 2.58\sqrt{al} \quad (7.17)$$

Here, X = distance from apex of slab corner to section of maximum stress along the corner bisector, cm

a = radius of wheel load distribution, cm

l = radius of relative stiffness, cm

7.4.2 Wheel Load Stresses

A. T. Goldbeck indicated that many concrete slabs failed at the corners. He derived a corner load formula due to a point load at the corner of the slab. Goldbeck's formula for stress due to corner load is given by :

$$S_c = \frac{3P}{h^2} \qquad (7.18)$$

Here, S_c = stress due to corner load, kg/cm^2

P = corner load assumed as a concentrated point load, kg

h = thickness of slab, cm

However the assumptions of unsupported corner and concentrated point load at corner have been later found to be severe resulting in very high thickness requirement of slabs.

Westergaard's stress equation for wheel loads

The cement concrete slab is assumed to be a homogeneous, thin elastic plate with subgrade reaction being vertical and proportional to the deflection.

The commonly used equations for theoretical computation of wheel load stresses have been given by Westergaard. He considered three typical regions of the cement concrete pavement slab for the analysis of stresses, as the interior, edge and the corner regions. The critical stresses S_i, S_e and S_c at the typical locations i.e. interior, edge and corner are given in Eq. 7.19 to 7.21.

Interior Loading

$$S_i = \frac{0.316\,P}{h^2}\,[4 \log_{10}(l/b) + 1.069] \qquad (7.19)$$

Edge Loading

$$S_e = \frac{0.572\,P}{h^2}\,[4 \log_{10}(l/b) + 0.359] \qquad (7.20)$$

Corner Loading

$$S_c = \frac{3\,P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)\right] \qquad (7.21)$$

Here,

S_i, S_e, S_c = maximum stress at interior, edge and corner loading, respectively, kg/cm^2

h = slab thickness, cm

P = wheel load, kg

a = radius of wheel load distribution, cm

l = radius of relative stiffness, cm (see Eq. 7.15)

b = radius of resisting section, cm (see Eq. 7.16)

Charts for stress computations

If the slab thickness h is to found for the allowable values of maximum stresses S_i, S_e and S_c trials are required for assumed values of h. Bradbury suggested a simplified procedure by expressing all equations in the general form

$$S = \frac{P}{h^2}\,Q \qquad (7.22)$$

He presented charts to find the values of *stress coefficients* Q, from the values of $\left(\frac{l}{b}\right)$ or $\left(\frac{a}{l}\right)$

Khanna et al have given a set of design charts in metric units, in Journal of I.R.C. Volume XXXIII-2 1970 based on Westergaard's equations for computation of wheel load stresses. These charts may be used to find the load stresses at interior edge and corner regions of the cement concrete slab, instead of using Westergaard's stress equations. There is a considerable saving in time in the computation of load stresses by the use of these charts.

Evaluation of wheel load stresses for design

Westergaard's wheel load stress equations for interior, edge and corner have been modified by various investigators based on their research work on cement concrete pavement slabs. The stresses at the edge and corner regions are generally found to be more critical for the design of rigid pavement for highways. The Indian Roads Congress recommends the following two formulas for the analysis of load stresses at the edge and corner regions and for the design of rigid pavements;

(i) Westergaard's edge load stress formula, modified by Teller and Sutherland for finding the load stress S_e in the critical edge region,

$$S_e = 0.529\,\frac{P}{h^2}\,(1 + 0.54\,\mu) \times (4 \log_{10} l/b + \log_{10} b - 0.4048) \qquad (7.23)$$

(ii) Westergaard's corner load stress analysis modified by Kelley for finding the load stress S_c at the critical corner region,

$$S_c = \frac{3\,P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \qquad (7.24)$$

where, S_e = load stress at the edge region, kg/cm^2

S_c = load stress at the corner region, kg/cm^2

P = design wheel load, kg

h = thickness of CC pavement slab, cm

μ = Poisson's ratio of the CC slab

E = modulus of elasticity of the CC, kg/cm^2

K = reaction modulus of pavement foundations (i.e., base course, sub-base course or subgrade), kg/cm^3

l = radius of relative stiffness, cm = $\left[\dfrac{E h^3}{12 K (1 - \mu^2)}\right]^{\frac{1}{4}}$

b = radius of equivalent distribution of pressure, cm,

b = a, when $\dfrac{a}{h} \geq 1.724$ and

b = $\sqrt{1.6 a^2 + h^2} - 0.675$ b, when $\dfrac{a}{h} \leq 1.724$

a = radius of load contact, cm (assumed circular in shape)

The above equations 7.23 and 7.24 for finding load stresses at the edge and corner regions are presented in the form of stress charts by the IRC and these are shown in Fig. 7.23 and 7.24. These charts are applicable for a particular set of design parameters only viz. : P = 5100 kg, a = 15 cm, E = 3×10^5 kg/cm^2, μ = 0.15; but different curves are given for different values of K between 6.0 and 30 kg/cm^3. The design curves are for slab thickness values, h = 14 to 25 cm. These stress charts are very handy and save considerable time when the stresses are to be evaluated for various trial thickness of the slab while designing a pavement.

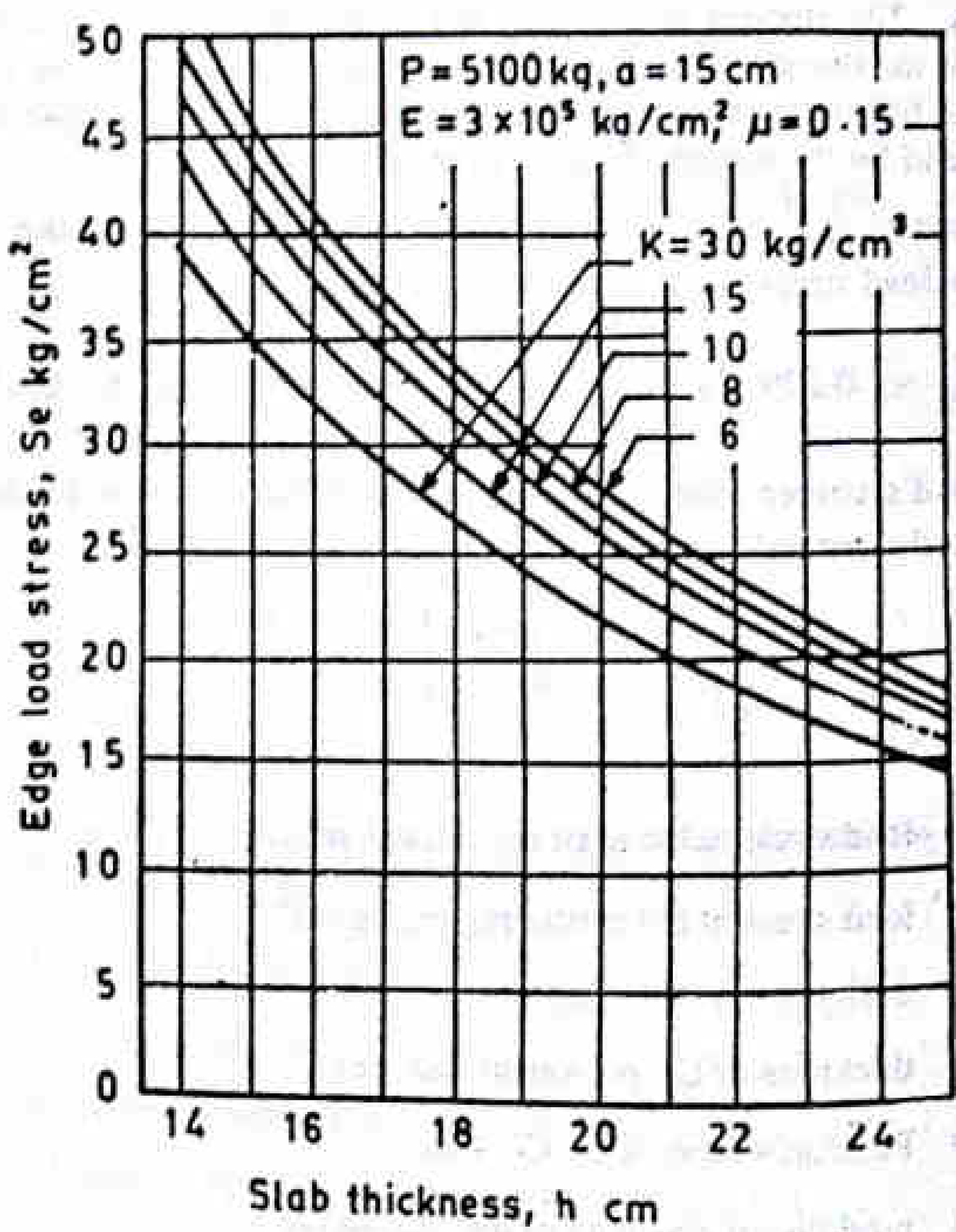


Fig. 7.23 Edge Load Stress Chart (IRC)

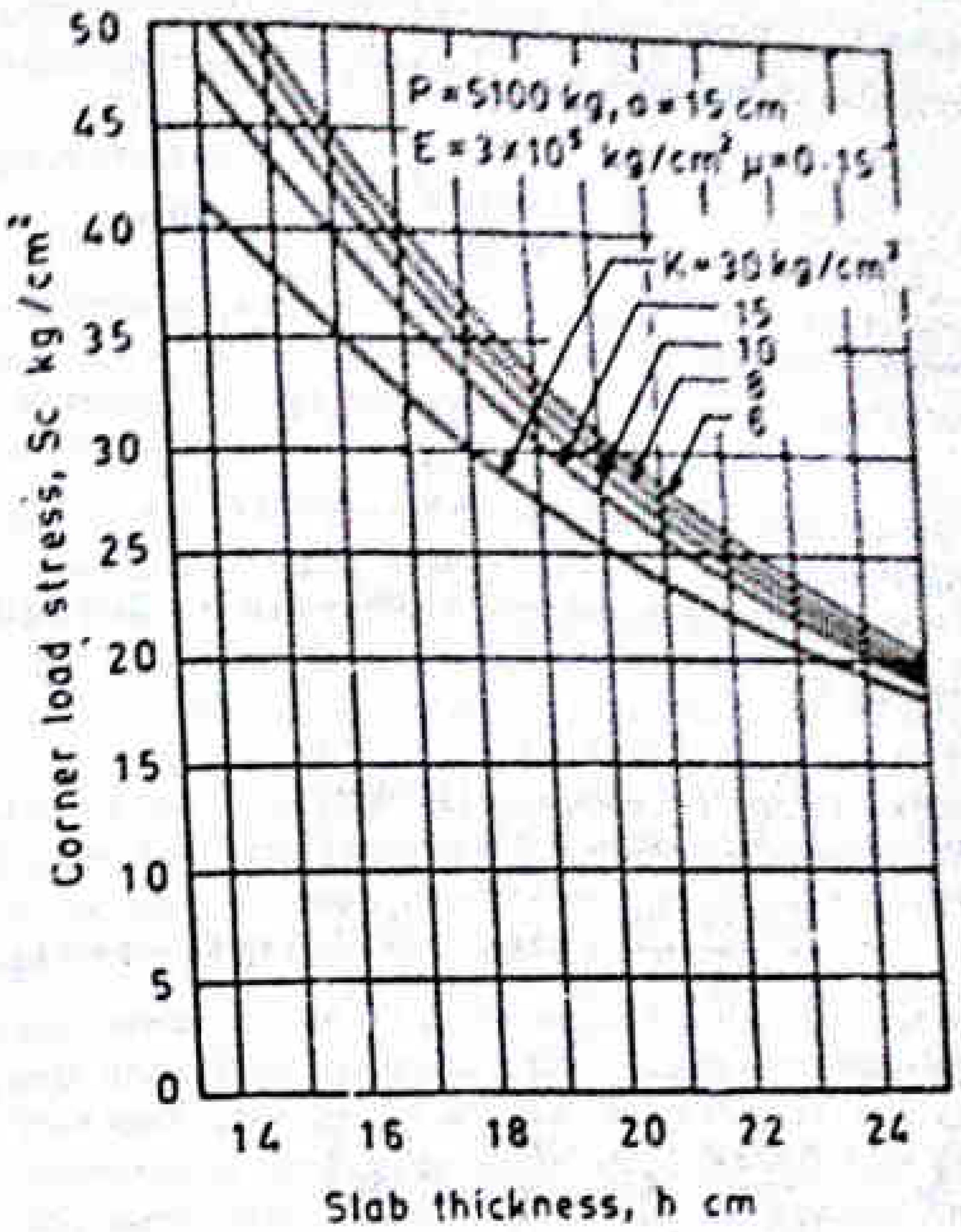


Fig. 7.24 Corner Load Stress Chart (IRC)

Load Stress Parameters

Wheel load P = 5100 kg,

Radius a = 15 cm

Elastic modulus of cement concrete E = 3×10^5 kg/cm^2,

Poisson's ratio μ = 0.15

Example 7.15

Calculate the stresses at interior, edge and corner regions of a cement concrete pavement using Westergaard's stress equations. Use the following data :

Wheel load, P = 5100 kg

Modulus of elasticity of cement concrete, E = 3.0×10^5 kg/cm^2

Pavement thickness, h = 18 cm

Poisson's ratio of concrete, μ = 0.15

Modulus of subgrade reaction, K = 6.0 kg/cm^3

Radius of contact area, a = 15 cm

Solution

Radius of relative stiffness (l) is given by

$$l = \left[\frac{E\,h^3}{12\,K\,(1-\mu^2)}\right]^{\frac{1}{4}} = \left[\frac{3.0\times10^5\times18^3}{12\times6\,(1-0.15^2)}\right]^{\frac{1}{4}} = 70.6 \text{ cm}$$

The equivalent of resisting section is given by

$$a/h = 15/8 = 0.833 < 1.74$$

$$b = \sqrt{1.6a^2 + h^2} - 0.675\,h$$

$$= \sqrt{1.6\times15^2 + 18^2} - 0.675\times18 = 14.0 \text{ cm}$$

Stress at the interior, (S_i)

$$S_i = \frac{0.316\,P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.069\right]$$

$$= \frac{0.316\times5100}{18^2}\left[4\log_{10}\left(\frac{70.6}{14.0}\right) + 1.069\right] = 19.3 \text{ kg/cm}^2$$

Stress at the Edge (S_e)

$$S_e = \frac{0.572\,P}{h^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0.359\right]$$

$$= \frac{0.572\times5100}{18^2}\,[4\times0.7027 + 0.359] = 28.54 \text{ kg/cm}^2$$

Stress at the Corner (S_c)

$$S_c = \frac{3\,P}{h^2}\left[2 - \left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]$$

$$= \frac{3\times5100}{18^2}\left[1 - \left(\frac{15\sqrt{2}}{70.6}\right)^{0.6}\right] = 24.27 \text{ kg/cm}^2$$

Example 7.16

A CC pavement of thickness 20 cm rests over a WBM base course with modulus of reaction 30 kg/cm^3. Find the load stresses at the edge and corner regions under a wheel load of 5100 kg unit IRC stress charts. (Assume a = 15 cm, E = 3 × 10^5 kg/cm^2 and μ = 0.15).

Solution

Refer edge load stress chart (Fig. 7.23). Using the curve for K = 30 kg/cm^3 corresponding to h = 20 cm, edge load stress S_e = 22.0 kg/cm^2.

Referring to corner load stress chart (Fig. 7.24), using curve K = 30 and corresponding to h = 20, corner load stress S_c = 25.5 kg/cm^2.

7.4.3 Temperature Stresses

Westergaard's Concept for Temperature Stresses

Temperature stresses are developed in cement concrete pavement due to variation in slab temperature. The variation in temperature across the depth of the slab is caused by *daily variation* whereas an overall increase or decrease in slab temperature is caused by *seasonal variation* in temperature.

During the day, the top of the pavement slab gets heated under the sun light when the bottom of the slab still remains relatively colder. The maximum difference in temperature between the top and bottom of the pavement slab may occur at some period after the mid-noon. This causes the slab to warp or bend, as the warping is resisted by the self weight of the slab, *warping stresses* are developed late in the evening, the bottom of the slab gets heated up due to heat transfer from the top and as the atmospheric temperature falls, the top of the slab becomes colder resulting in warping of the slab in the opposite direction and there is a reversal in warping stresses at the different regions of the slab. Thus the daily variation in temperature causes warping stresses in reverse directions at the corner, edge and interior regions of the slab.

During summer season as the mean temperature of the slab increases, the concrete pavement expands towards the expansion joints. Due to the frictional resistance at the interface (which depends upon the self weight of the slab and the coefficient of friction at the interface), compressive stress is developed at the bottom of the slab as it tends to expand. Similarly during winter season, the slab contracts causing tensile stress at the bottom due to the frictional resistance again opposing the movement of the slab. Thus *frictional stresses* are developed due to seasonal variation in temperature. The frictional stress will be zero at the free ends and at expansion joints and increases upto a maximum value towards the interior and there-after remains constant.

Temperature thus tends to produce two types of stresses in a concrete pavement. These are

(i) warping stresses and

(ii) frictional stresses.

Warping stresses

Whenever the top and bottom surfaces of a concrete pavement simultaneously possess different temperatures, the slab tends to warp downward or upward inducing warping stresses. See Fig. 8.18, under Highway Construction chapter.

The difference in temperature between the top and bottom of the slab depends mainly on the slab thickness and the climatic conditions of the region.

By the time the top temperature increases to t_1 degrees, the bottom temperature may be only t_2 degrees and the difference between the top and bottom of the slab would be $(t_1 - t_2)$ = t degrees.

Assuming straight line variation in temperature across the pavement depth, the temperature at mid depth or average temperature of slab would be $(t_1 + t_2)/2$.

If the slab has no restraint then the unit elongation of the top fibres and also the contraction of the bottom fibre due to relative temperature condition, each would be equal to Eet/2 where e is the thermal coefficient of concrete. Westergaard worked out the stresses due to the warping of concrete slabs.

Now introducing the effect of Poisson's ratio the stresses at the interior, region in longitudinal and transverse directions as given by *Bradbury* are expressed by the following equations :

$$St_{(i)} = \frac{Eet}{2}\left[\frac{C_x + \mu C_y}{1-\mu^2}\right] \quad (7.25)$$

Here, $St_{(i)}$ = warping stress at interior, kg/cm^2

E = modulus of elasticity of concrete, kg/cm^2

e = thermal coefficient of concrete per °C

t = temperature difference between the top and bottom of the slab in degree C

C_x = coefficient based on L_x/l in desired direction

C_y = coefficient based on L_y/l in right angle to the above direction

μ = Poisson's ratio (may be taken as 0.15)

L_x and L_y are the dimensions of the slab considering along X and Y directions along the length and width of slab.

The values of the warping stress coefficients C_x and C_y for cement concrete pavement are taken from the chart developed by Bradbury. See Fig. 7.25. The warping stress at the edge region is given by :

$$St_{(e)} = \frac{C_x\, E.e.t}{2} \quad (7.26a)$$

or

$$= \frac{C_y\, E.e.t}{2} \text{ (whichever is higher)} \quad (7.26b)$$

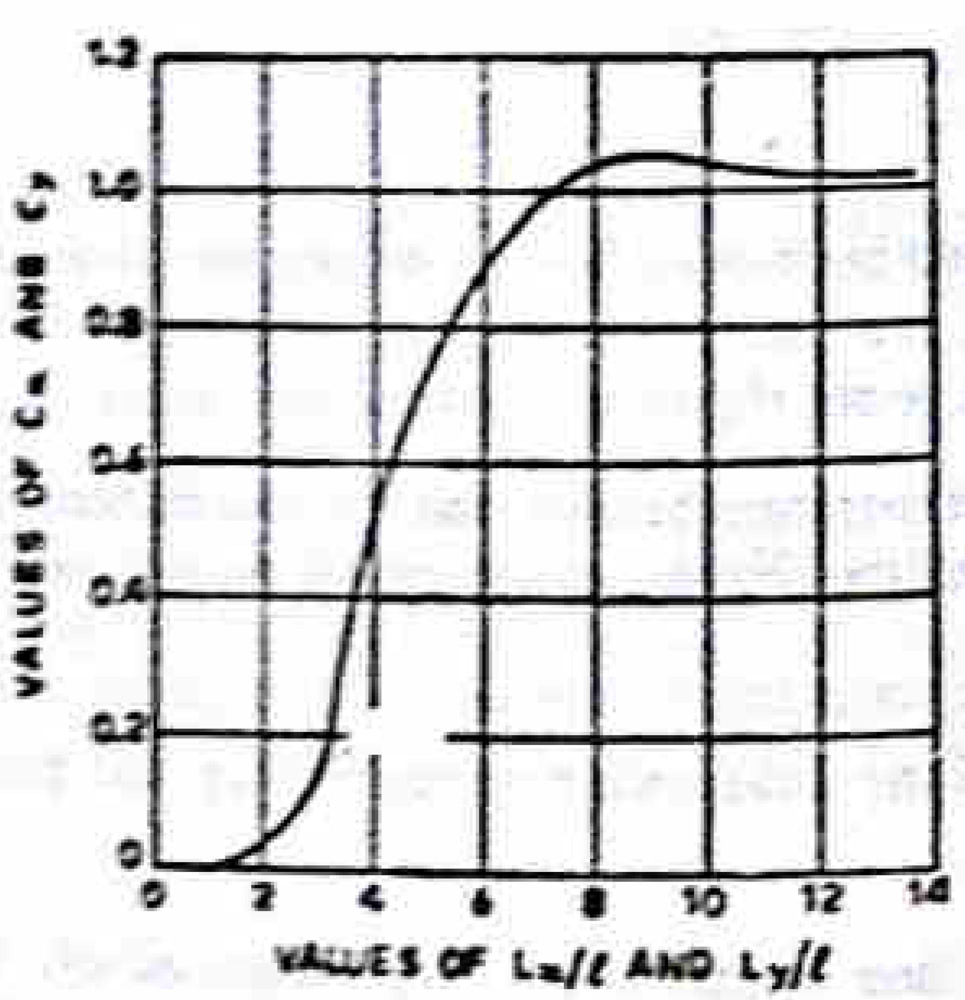


Fig. 7.25 Warping Stress Coefficient

For corner region, warping stress is given by :

$$St_{(c)} = \frac{E.e.t}{3(1-\mu)}\sqrt{\frac{a}{l}} \quad (7.27)$$

Here, a is radius of contact and l is the radius of relative stiffness.

Example 7.17

Determine the warping stresses at interior, edge and corner regions in a 25 cm thick concrete pavement with transverse joints at 11 m interval and longitudinal joints at 3.6 m intervals. The modulus of subgrade reaction (K) is 6.9 kg/cm^3. Assume temperature differential for day conditions to be 0.6°C per cm slab thickness. Assume radius of loaded area as 15 cm for computing warping stress at the corner. Additional data are given below :

$$e = 10 \times 10^{-6} \text{ per °C}$$

$$E = 3 \times 10^5 \text{ kg/cm}^2$$

$$\mu = 0.15$$

Solution

Calculate the radius of relative stiffness

$$l = \left[\frac{Eh^3}{12K(1-\mu^2)}\right]^{\frac{1}{4}}$$

$$= \left[\frac{3\times10^5\times25^3}{12\times6.9(1-0.15^2)}\right]^{0.25} = 87.2 \text{ cm}$$

$$\frac{L_x}{l} = \frac{1100}{87.2} = 12.61$$

From Fig. 7.25, C_x = 1.03

Also $\frac{L_y}{l} = \frac{360}{87.2} = 4.13,\ C_y = 0.55$

$$t = 25 \times 0.6 = 15 \text{ °C}$$

Interior warping stress from Equation 7.25

$$St_{(i)} = \frac{3\times10^5\times10\times10^{-6}\times15}{2}\left[\frac{1.03+0.15\times0.55}{1-0.15^2}\right]$$

$$= 25.61 \text{ kg/cm}^2$$

Longitudinal edge stresses due to warping from Eq. 7.26 (Using C_x value, as it is higher than C_y)

$$St_{(e)} = \frac{1.03\times3\times10^5\times10\times10^{-6}\times15}{2} = 23.18\ kg/cm^2$$

Warping stress at the corner region from Eq. 7.27.

$$St_{(c)} = \frac{3\times10^5\times10\times10^{-6}\times15}{3(1-0.15)}\sqrt{\frac{15}{87.2}} = 6.36\ kg/cm^2$$

Thomlinson's Temperature Stress Analysis

J. Thomlinson in 1940 provided an analytical approach for temperature stress computations. From actual measurements of temperature in cement concrete pavements using *Thermocouples*, it has been seen that the temperature gradient across the slab thickness is infact curvilinear as against in the assumption of straight line variation by Westergaard. Thomlinson developed an analysis which yields results more close to the experimental data and lower than Westergaard's equations. Details of this analysis are not discussed here.

Frictional stresses

Due to uniform temperature rise and fall in the cement concrete slab, there is an overall expansion and contraction of the slab. Since the slab is in contact with soil subgrade or the sub-base, the slab movements are restrained due to the friction between the bottom layer of the pavement and the soil layer. This frictional resistance therefore tends to prevent the movements thereby inducing the frictional stress in the bottom fibre of the cement concrete pavement. Stresses in slabs resulting due to this phenomenon vary with slab length. In short, slab stress induced due to this is negligibly small whereas in long slabs, which would undergo movements of more than 0.15 cm, higher amount of frictional stress develops.

Equating, total force developed in the cross section of concrete pavement due to movement and frictional resistance due to subgrade restraint in half the length of the slab,

$$S_f \times h \times B \times 100 = B \times \frac{L}{2} \times \frac{h}{100} \times W \times f$$

$$S_f = \frac{WLf}{2\times10^4} \quad (7.28)$$

Here S_f = unit stress developed in cement concrete pavement, kg/cm^2

W = unit weight of concrete, kg/cm^3 (about 2400 kg/m^3)

f = coefficient of subgrade restraint (maximum value is about 1.5)

L = slab length, metre

B = Slab width, metre

7.4.4 Combination of Stresses

It is necessary to consider the conditions under which the various stresses in cement concrete pavements would combine to give the most critical combinations.

The following conditions are considered to provide the critical combinations :

(i) *During summer* : The critical combinations at interior and edge regions during mid day occurs when the slab tends to warp downward. During this period maximum tensile stress is develop at the bottom fibre due to warping and this is cumulative with the tensile stress due to the loading. However the frictional stress is compressive during expansion. The load stress at edge region is higher than the interior.

Critical combination of stresses = load stress + warping stress − frictional stress), *at edge region.*

(ii) *During winter* : The critical combination of stresses at the above regions occurs at the bottom fibre when the slab contracts and the slab warps downward during the mid day. The frictional stress is tensile during contraction.

The critical stress combination = (load stress + warping stress + frictional stress), *at edge region.*

Since the differential temperature t is of lower magnitude during winter than in summer, the combination (i) may be worst for most of the regions in this country.

(iii) At corner region, the critical combination occurs at the top fibre of the slab, when the slab warps upwards during the mid nights. There is no frictional stress at the corner region.

The critical stress combination = (load stress + warping stress), *at corner regions.*

Example 7.18

A CC pavement slab of thickness 20 cm is constructed over a granular sub-base having modulus of reaction 15 kg/cm^2. The maximum temperature difference between the top and bottom of the slab during summer day and night is found to be 18°C. The spacing between the transverse contraction joint is 4.5 m and that between longitudinal joints is 3.5 m. The design wheel load is 5100 kg, radius of contact area is15 cm, E value of CC is 3×10^5 kg/cm^2, Poisson's ratio is 0.15, and coefficient of thermal expansion of CC is 10×10^{-6} per °C and friction coefficient is 1.5. Using the edge and corner load stress charts given by the IRC and the chart for the warping stress coefficient, find the worst combination of stresses at the edge.

Solution

(i) Edge Region

(a) Edge load stress from chart (Fig. 7.23).

for h = 20 cm and K = 15 kg/cm^3, S_e = 24.0 kg/cm^2

(b) Warping stress at edge :

$$\text{Radius of relative stiffness } l = \left[\frac{3\times10^5\times20^3}{12\times15(1-0.15^2)}\right]^{\frac{1}{4}} = 60.8\ cm$$

Length of slab L_x = 4.5 m = 450 cm

Warping stress coefficient, C_x from Fig. 7.25, at

$$\frac{L_x}{l} = \frac{450}{60.8} = 7.4,\ C_x = 1.02$$

Similarly at $\frac{L_y}{l} = \frac{350}{60.8} = 5.75,\ C_y = 0.87;$

$$t = 18\ °C$$

Maximum warping stress at edge,

$$S_{te} = \frac{Eet}{2} \cdot C_x = \frac{1}{2} \times 3 \times 10^5 \times 10 \times 10^{-6} \times 18 \times 1.02 = 27.54\ kg/cm^2$$

(c) Frictional stress :

From Eq. 7.28, $S_f = \frac{W.L_x.f}{2 \times 10^4} = \frac{2400 \times 4.5 \times 1.5}{2 \times 10^4} = 0.81\ kg/cm^2$

(d) Combined stress at edge region :

Critical combination of stress during summer mid-day = load stress + warping stress – frictional stress

$$= 24.0 + 27.54 - 0.81 = 50.73\ kg/cm^2$$

(ii) Corner Region

(a) Load stress :

From chart 7.24 for h = 20 and K = 15,

$$S_c = 28.0\ kg/cm^2$$

(b) Max. warping stress :

From Eq. 7.27,

$$St_c = \frac{E.e.t}{3(1-\mu)} \sqrt{\frac{a}{l}} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 18}{3(1-0.15)} \sqrt{\frac{15}{60.8}} = 9.15\ kg/cm^2$$

(c) Frictional stress : This is zero at corner region

(d) Combined stress at the corner region :

The critical combination of stress in summer mid-night = load stress + warping stress

$$= 28.0 + 9.15 = 37.15\ kg/cm^2$$

(It may be noted that the critical combination of stresses at the edge region is higher than that at the corner under the identical condition of pavement, load and temperature).

7.4.5 Design of Joints in Cement Concrete Pavements

Various types of joints provided in cement concrete pavements to reduce the temperature stresses are expansion joint, contraction joints and warping joints. If expansion and contraction joints are properly designed and constructed, there is no need of providing warping joints, in addition. Expansion joint spacing is designed based on the maximum temperature variations expected and the width of joint. The contraction

joint spacing design is governed by the anticipated frictional resistance and allowable tensile stress in concrete during the initial curing period, or the amount of reinforcement if any. The spacing between the expansion joints is so adjusted that the contraction joints have equal spacing. *Dowel bars* are provided at expansion joints and some times at contraction joints also. The size and spacing of the dowel bars are designed and are also governed by standard specification based on practical considerations. Longitudinal joints in cement concrete pavements are constructed with suitable *tie bars*. The design considerations include diameter, spacing and length of the bars.

Spacing of expansion joint

The width or the gap in expansion joint depends upon the length of slab. Greater the distance between the expansion joints, the greater is the width required of the gap for expansion. The use of wide expansion joint space should be avoided as it would be difficult to keep them properly filled in when the gap widens during winter season. The dowels would develop high bending and bearing stresses with wider openings. It is recommended not to have a gap more than 2.5 cm in any case. The IRC has recommended that the maximum spacing between expansion joints should not exceed 140 m for rough interface layer.

If 'δ'' is the maximum expansion in a slab of length L_e with a temperature rise from T_1 to T_2.

$\delta' = L_e C (T_2 - T_1)$ where C is the thermal expansion of concrete per degree rise in temperature.

The joint filler may be assumed to be compressed up to 50 percent of its thickness and therefore, the expansion joint gap should be twice the allowable expansion in concrete, i.e., 2δ'. From the relation given above, if δ' is half the joint width, the spacing of expansion joint L_e is given by the equation :

$$L_e = \frac{\delta'}{100\,C\,(T_2 - T_1)} \qquad (7.29)$$

Example 7.19

The width of expansion joint gap is 2.5 cm in a cement concrete pavement. If the laying temperature is 10°C, and the maximum slab temperature in summer is 54°C, calculate the spacing between expansion joints. Assume coefficient of thermal expansion of concrete as 10×10^{-6} per °C.

Solution

$$\delta' = \frac{2.5}{2}$$

$$= 1.25\ cm$$

$$T_2 - T_1 = 54 - 10 = 44°C$$

$$L = \frac{1.25}{100 \times 10 \times 10^{-6} \times 44}$$

$$= 28.5\ m$$

Spacing of contraction joints

The slab contracts due to the fall in slab temperature below the construction temperature. Also during the initial curing period, shrinkage occurs in cement concrete. This movement is resisted by the subgrade drag or friction between the bottom fibre of the slab and the subgrade; see Fig. 7.26. If L_c is the slab length in metre, the maximum stress occurs at half the length.

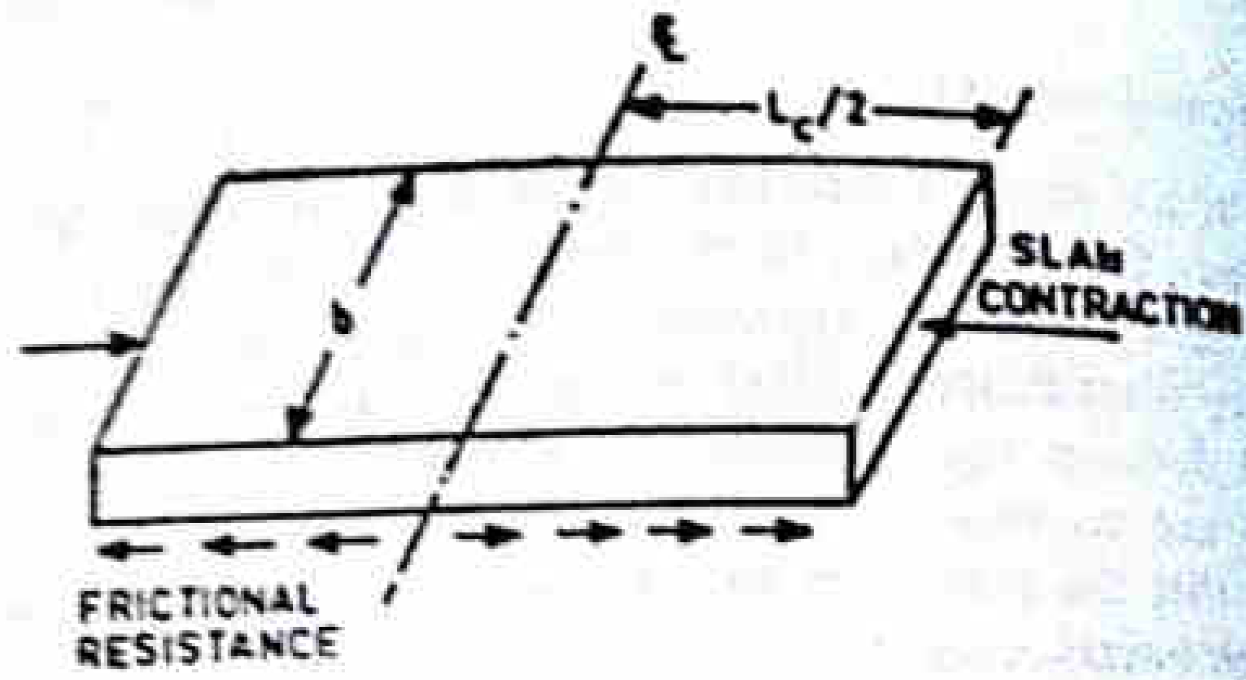


Fig. 7.26 Slab Contraction and Frictional Resistance

Total frictional resistance upto distance $L_c/2$ = $W \times b \times (L_c/2) \times (h/100) \times f$

Allowable tension in cement concrete = $S_c \times h \times b \times 100$

Equating the above two values,

$$\frac{b\,L_c\,h\,f}{200} = 100\,S_c\,h\,b$$

Length of slab to resist the frictional drag, i.e., spacing of contraction joints,

$$L_c = \frac{2\,S_c}{Wf} \times 10^4 \qquad (7.30)$$

Here, L_c = slab length or spacing between contraction joints, m

h = slab thickness, cm

f = coefficient of friction, (maximum value is about 1.5)

W = unit weight of cement concrete, kg/m^3 (2400 kg/m^3)

S_c = allowable stress in tension in cement concrete, kg/cm^2 (0.8 kg/cm^2)

Since the contraction or shrinkage cracks develop mainly during initial period of curing, a very low value of S_c is considered in design. The permissible stress is generally kept as low as about 0.8 kg/cm^2.

Spacing of Contraction Joints when Reinforcement is provided

If it is assumed that the reinforcement takes the entire tensile force in the slab, caused by the frictional resistance of subgrade and *hair cracks* are allowed, then

$$W \times b \times \frac{L_c}{2} \times \frac{h}{100} \times f = S_s\,A_s$$

$$L_c = \frac{200\,S_s\,A_s}{b\,h\,W\,f} \qquad (7.29)$$

Here, A_s = total area of steel, cm^2 across the slab width

L_c = spacing between contraction joints, m

b = slab width, m

h = slab thickness, cm

W = unit weight of cement concrete, kg/m^3 (2400)

f = coefficient of friction (1.5 max)

S_s = allowable tensile stress in steel, kg/cm^2 (1400)

Example 7.20

Determine the spacing between contraction joints for 3.5 meter slab width having thickness of 20 cm and f = 1.5, for the following two cases :

(i) for plain cement concrete, allowable S_c = 0.8 kg/cm^2

(ii) for reinforcement cement concrete, 1.0 cm dia. Bars at 0.30 m spacing

Solution

Case (i) For Plain Cement Concrete Slab (without reinforcement)

Assume unit weight of CC, W = 2400 kg/m^3

Using Eq. 7.30 spacing between contraction joints

$$L_c = \frac{2S_c}{Wf} \times 10^4 = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.5} = 4.44 \text{ m}$$

Case (ii) For Reinforced Cement Concrete Slab

Total cross sectional area of steel, A_s in one direction along the slab width

$$A_s = \frac{3.5 \times \pi \times 1.0^2}{0.3 \times 4} = 9.16 \text{ cm}^2$$

Using Eq. 7.29, spacing between contraction joints,

$$L_c = \frac{200\,S_s\,A_s}{b\,h\,W\,f} = \frac{200 \times 1200 \times 9.16}{3.5 \times 20 \times 2400 \times 1.5} = 8.72 \text{ m}$$

Example 7.21

The maximum increase in temperature is expected to be 26°C after the construction of a CC pavement. If the expansion joint gap is 2.2 cm, design the spacings between the expansion and contraction joints. Assume plain cement concrete construction, with thermal coefficient = 10×10^{-6} per°C, unit weight = 2400 kg/m^3, allowable stress in tension during initial period of curing = 0.8 kg/cm^2 and the coefficient of friction of the interface = 1.4.

Solution

Spacing between contraction joints in plain CC pavement,

$$L_c = \frac{2S_c}{Wf} \times 10^4 = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.4} = 4.76 \text{ m}$$

Maximum spacing suggested by the IRC is 4.5 m for plain CC pavements and so adopt $L_c = 4.5$ m

Spacing between expansion joints,

$$L_e = \frac{\delta'}{100\,C\,(T_2 - T_1)}$$

$$= \frac{1.1}{100 \times 10 \times 10^{-6} \times 26} = 42.3 \text{ m}$$

Therefore provide spacing of expansion joints = 9 × 4.5 = 40.5 m

(As 10 × 4.5 = 45.0 m which is higher than 42.3 m, expansion joints are provided after eight contraction joints or after the ninth slab).

Design of Dowel Bar

Dowel bars of expansion joints are mild steel round bars of short length. Half length of this bar is bonded in one cement concrete slab and the remaining portion is embedded in adjacent slab, but is kept free for the movement during expansion and contraction of the slab. The dowel bars allow opening and closing of the joint, maintaining the slab edges at the same level, and the load transference is effected from one slab to the other. This is explained below :

If dowel bars are not provided at the transverse joint (as in Fig. 7.27-a) the loaded slab would deflect by say, dx_1 under load P. The adjacent slab across the expansion joint does not participate in load bearing and it does not deflect at all.

When the joint is provided with dowel bar system and the load P is applied on the first slab under the same conditions, the loaded slab deflects by the magnitude say dx_2. (See Fig. 7.27-b) and the adjacent slab also undergoes a deflection, dx_3 due to the dowel bar transferring part of the load. Theoretically, if the dowel bars are rigidly embedded in the concrete slabs on both sides, dx_2 should be equal to dx_3. It has however been observed that $dx_3 < dx_2$ and their ratio depends upon the thickness of the slab, size and diameter of dowels and their spacing.

It may be noted that $dx_1 > dx_2$ or dx_3. Accordingly, the stress caused in loaded slab is greater when there are no dowels than the slab joint with dowel bars. This is logical as the stress magnitudes are related with deflections. The pressure distribution along the dowel bar under the load on one slab is illustrated in Fig. 7.27 c as per *Friberg's* analysis. Points of pressure reversal a, would determine the criterion for determining the length of dowel bars.

In the design of dowels, the load transference is worked out considering the capacity of the dowel system. The capacity depends upon variable like, pavement thickness, subgrade modulus, the relative stiffness and spacing and size of dowels.

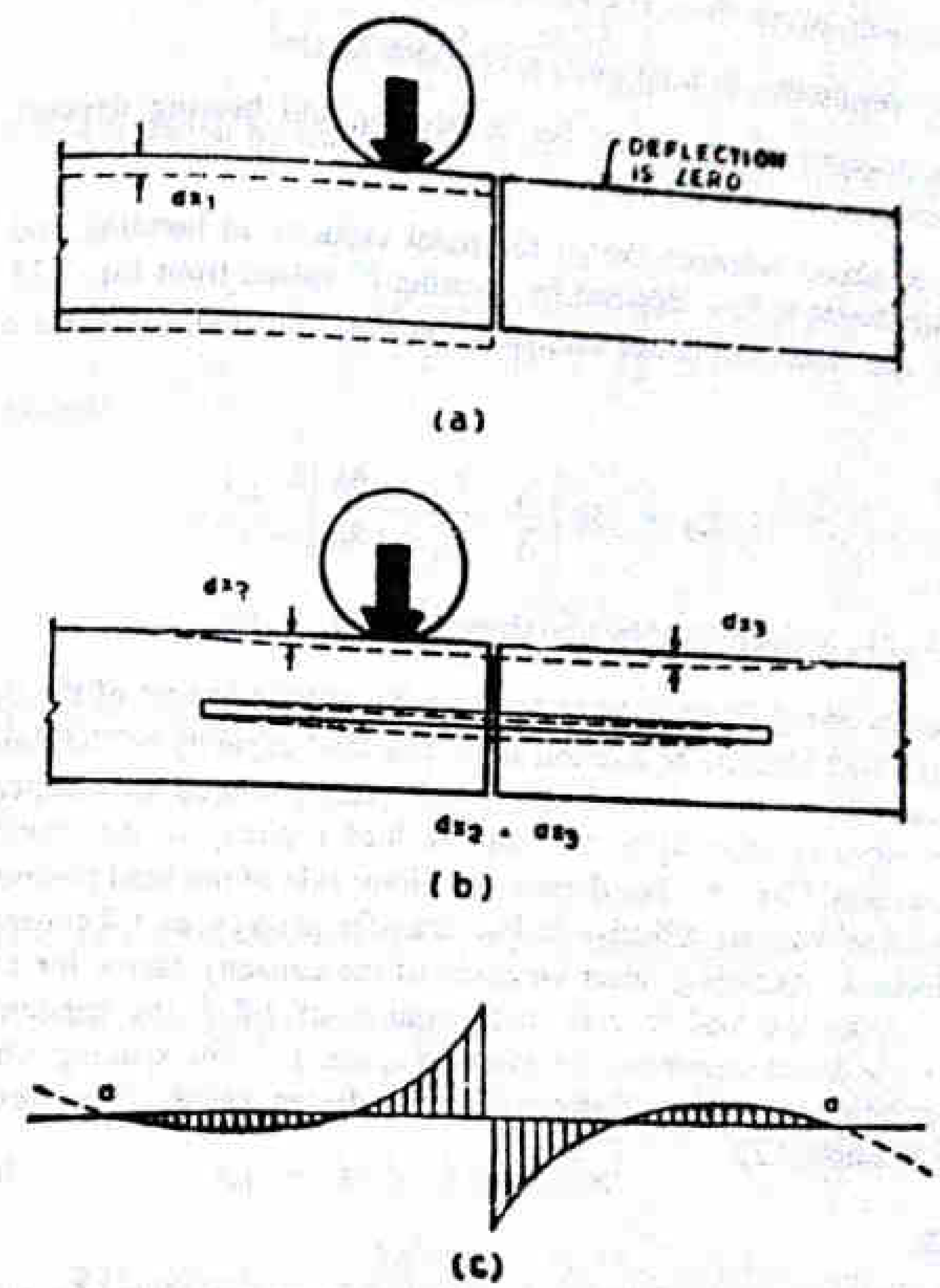


Fig. 7.27 Functioning of Dowel Bar

The IRC recommends that dowel bar system may be designed on the basis of Bradbury's analysis for load transfer capacity of a single dowel bar in shear, bending and bearing in concrete. These values are given below :

For shear in the bar, $P' = 0.785\, d^2 F_s$ (7.32)

For bending in the bar, $P' = \dfrac{2 d^3 F_f}{L_d + 8.8\delta}$ (7.33)

For bearing on concrete, $P' = \dfrac{F_b L_d^2 . d}{12.5 (L_d + 1.5\delta)}$ (7.34)

where, P' = load transfer capacity of a single dowel bar, kg

d = diameter of dowel bar, cm

L_d = total length of embedment of dowel bar, cm

δ = joint width, cm

F_s = permissible shear stress in dowel bar, kg/cm²

F_f = permissible flexural stress in dowel bar kg/cm^2

F_b = permissible bearing stress in concrete, kg/cm^2

The load capacity of the dowel bar in bending and bearing depend on the total embedded length L_d on both the slabs.

In order to obtain balanced design for equal capacity in bending and bearing, the length of embedment is first obtained by equating P' values from Eq. 7.33 and 7.34 for the assumed joint width and dowel diameter. On simplification, the value of L_d is given by

$$L_d = 5d\left[\frac{F_f}{F_b}\times\frac{L_d+1.5\delta}{L_d+8.8\delta}\right]^{\frac{1}{2}} \quad (7.35)$$

The value of L_d is determined by trial from Eq. 7.35.

The minimum dowel length is taken as ($L_d + \delta$), and the lowest of the three values of P' taken as the load capacity of a dowel bar. The load capacity of the dowel system or group is assumed to be 40% of the design wheel load. The required load capacity factor of the dowel group is obtained by dividing the load capacity of the group by the load capacity of one dowel bar, P'. The distance on either side of the load position upto which the group of dowel bars are effective in load transfer is taken as 1.8 times the radius of relative stiffness, l. Assuming linear variation of the capacity factor for a single dowel bar from 1.0 under the load to zero at a distance of 1.8 l, the capacity factors are calculated for the dowel system for the assumed spacings. The spacing which conforms to the required capacity factor is selected as the design value. The design steps are illustrated in Example 7.22.

Example 7.22

Design the size and spacing of dowel bars at the expansion joints of a cement concrete pavement of thickness 25 cm with radius of relative stiffness 80 cm, for a design wheel load of 5000 kg. Assume load capacity of the dowel system as 40% of the design wheel load. Joint width is 2.0 cm, permissible shear and flexural stresses in dowel bar are 1000 and 1400 kg/cm^2 respectively and permissible bearing stress in CC is 100 kg/cm^2.

Solution

Length of Dowel Bar, L

Assume the diameter of the dowel bar, d = 2.5 cm

$P = 5000$ kg $H = 25$ cm

$l = 80$ cm $\delta = 2$ cm

$F_s = 1000$ kg/cm^2 $F_f = 1400$ kg/cm^2

$F_b = 100$ kg/cm^2

For equal capacity of dowel bar in bending and bearing (Eq. 7.35),

$$L_d = 5\times 2.5\left[\frac{1400}{100}\times\frac{L_d+1.5\times 2}{L_d+8.8\times 2}\right]^{\frac{1}{2}}$$

$$= 12.5\left[14\times\frac{L_d+3}{L_d+17.6}\right]^{\frac{1}{2}}$$

Solution of this equation by trial method is simple. Therefore as a first trial assume $L_d = 45$ cm.

$$L_d = 12.5\left[14\times\frac{45+3}{45+17.6}\right]^{\frac{1}{2}} = 40.95$$

Which is less than 45

Assume $L_d = 40.5$

$$L_d = 12.5\left[14\times\frac{40.5+3}{40.5+17.6}\right]^{\frac{1}{2}} = 40.47$$

Therefore total length of embedment, $L_d = 40.5$ cm

Minimum length of dowel bar required, $L = L_d + \delta$

$= 40.5 + 2.0 = 42.5$ cm

Therefore provide 2.5 cm diameter dowel bars of length 45 cm.

Load Transfer Capacity of Single Dowel Bar, P'

$$P'(\text{shear}) = 0.785\,d^2 F_s = 0.785\times 2.5^2\times 1000 = 4906 \text{ kg}$$

Actual value of $L_d = 45.0 - 2.0 = 43$ cm

$$P'(\text{bending}) = \frac{2d^3 F_f}{L_d+8.8\delta} = \frac{(2\times 2.5^3\times 1400)}{(43+8.8\times 2)} = 722 \text{ kg}$$

$$P'(\text{bearing}) = \frac{F_b L_d^{\,2}.d}{12.5(L_d+1.5\delta)} = \frac{100\times 43^2\times 2.5}{12.5(43+3)} = 804 \text{ kg}$$

Taking the lowest of the three values for design, load capacity of a dowel bar, P' (design) = 722kg.

Required Load Capacity Factor

Load capacity of the dowel group = 40% of P = 0.4 × 5000 = 2000 kg

Required capacity factor for dowel group= $\frac{2000}{722} = 2.77$

Spacing of Dowel Bars

Effective distance upto which there is load transfer

$= 1.8\,l = 1.8\times 80 = 144$ cm

Assuming a trial spacing of 35 cm between the dowel bars, the capacity factor available for the group

$$= 1 + \frac{144-35}{144} + \frac{144-70}{144} + \frac{144-105}{144} + \frac{144-140}{144}$$

$$= 5 - \frac{350}{144} = 2.57$$

This value of capacity factor available is less than the capacity factor required, i.e. 2.77. Therefore assuming a closer spacing of 30 cm, capacity factor available,

$$= 1 + \frac{144-30}{144} + \frac{144-60}{144} + \frac{144-90}{144} + \frac{144-120}{144} = 2.92$$

As this is greater than the required capacity factor of 2.77, the spacing of the dowel bars is adequate. Therefore provide 2.5 cm dia. Dowel bars of total length 45 cm at 30 cm spacing.

Design of tie bars

Tie bars are used across the longitudinal joints of cement concrete pavements. Tie bars ensure two adjacent slabs to remain firmly together. These bars are not designed to act as load transfer devices. Tie bars are thus designed to withstand tensile stresses, the maximum tensile force in tie bars being equal to the force required to overcome friction force between the bottom of the adjoining pavement slab and the soil subgrade. The force is considered from the joint location to the subsequent joint or free edge. See Fig. 7.28.

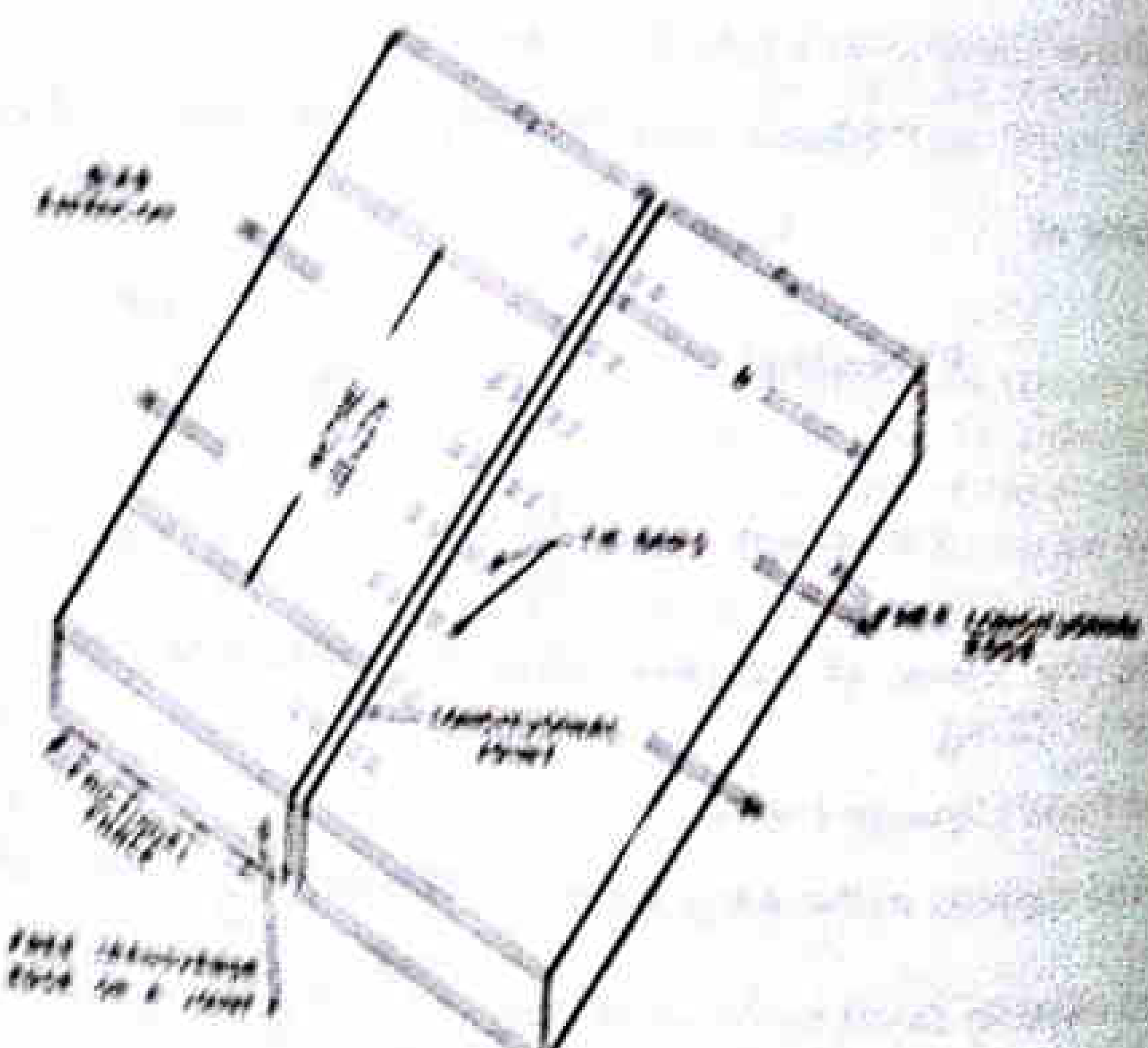


Fig. 7.28 Functioning of Tie Bars

Diameter and Spacing

The diameter and spacing of the tie bars are calculated as explained under :

Area of steel per meter length of joint is obtained by equating the total subgrade friction to the total tension developed in the tie bars.

Thus considering one meter length of the joint,

$$A_s S_s = b \frac{h}{100} W f$$

$$A_s = \frac{b f h W}{100 S_s} \qquad (7.36)$$

where A_s = area of steel per metre length of joint, cm^2

b = distance between the joint and nearest free edge, m

h = thickness of pavement, cm

f = Coefficient of friction between pavement and subgrade (may be taken as 1.5 for rough interface)

W = unit weight of cement concrete, kg/m^3 (2400 kg/m^3)

S_s = allowable working stress in tension for steel, kg/cm^2 (1400 kg/cm^2)

Assuming a suitable diameter of the tie bar (0.8 to 1.5 cm) the spacing of the tie bar can be found to provide the area of steel A_s cm^2 per meter length of the slab.

Length of Tie Bar

The total length of the bar should be atleast twice the length of embedment required on each slab to develop a bond strength equal to the working stress of the steel.

This is obtained from the consideration that the total tensile force developed in tie bar should not exceed the bond strength between tie bar and the concrete. Therefore considering one side of the longitudinal joints,

$$a_s S_s = \frac{L_t}{2} P S_b$$

$$L_t = \frac{2 a_s S_s}{P S_b} \qquad (7.37)$$

Substituting $a_s = \pi d^2/4$ and $P = \pi d$,

$$L_t = \frac{d S_s}{2 S_b}$$

Hence total length of tie bar $= L_t = \frac{2 a_s S_s}{P S_b} = \frac{d S_s}{2 S_b}$ (7.38)

Here, $L_t/2$ = length of tie bar on one side of slab, cm or half length of tie bar

S_s = allowable stress in tension, kg/cm^2 (1400 kg/cm^2)

S_b = allowable bond stress in concrete, kg/cm^2 (this is taken as 24.6 kg/cm^2 for deformed bars and 17.5 kg/cm^2 in plain tie bars).

a_s = cross sectional area of one tie bar, cm^2

P = perimeter of tie bar, cm

d = diameter of tie bar, cm

Example 7.23

As cement concrete pavement has a thickness of 18 cm and has two lanes of 7.2 m with a longitudinal joint along the centre. Design the dimensions and spacing of the tie bar. Use the following data :

Allowable working stress in tension, S_s = 1400 kg/cm²

Unit weight of concrete, W = 2400 kg/m³

Coefficient of friction, f = 1.5

Allowable bond stress in deformed bars in concrete, S_b = 24.6 kg/cm²

Solution

Area of steel per metre of longitudinal joint is given by Eq. 7.36.

$$b = \frac{7.2}{2} = 3.6 \text{ m}; h = 18 \text{ cm},$$

$$A_s = \frac{b\,f\,h\,W}{100\,S_s} = \frac{3.6\times1.5\times18\times2400}{100\times1750} = 1.333 \text{ cm}^2/\text{m}$$

Using 1 cm diameter bars having area of cross section $a_s = 0.785$ cm², the spacing of the tie bars is given by

$$\text{Spacing} = \frac{100\times0.785}{1.333} = 58.9 \text{ cm, (say 58 cm)}$$

Using 1 cm diameter tie bars, the length L_t is obtained from Eq. 7.38 and is given by:

$$L_t = \frac{d\,S_s}{2\,S_b} = \frac{1\times1400}{2\times24.6} = 28.5 \text{ cm}$$

Total length of tie bar = 30 cm, say

Use 1 cm diameter dia bars of length 30 cm at 50 to 58 cm c/c.

7.4.6 IRC Recommendations for Design of Concrete Pavements

(a) Design Parameters

(i) The design wheel load is taken as 5100 kg with equivalent circular area of 15 cm and a tyre inflation pressure ranging from 6.3 to 7.3 kg/cm². The traffic volume is projected for 20 years period after construction using the relation :

$$A_d = P'\,[1 + r]^{(n+20)} \tag{7.39}$$

where A_d = number of commercial vehicles per day (laden weight > 3 tonnes)

P' = number of commercial vehicles per day at last count.

r = annual rate of increase in traffic intensity (may be taken as 7.5% for rural roads if data is not available).

n = number of years between the last traffic count and the commissioning of new cement concrete pavement.

The traffic intensity so obtained is classified and adjustment for the pavement design thickness is made as given in the Table below :

Traffic Classification	Design traffic intensity, A_d (no. of vehicles of wt > 3 tonnes, per day) at the end of design life	Adjustment in design thickness of cc pavement, cm
A	0 to 15	−5
B	15 to 45	−5
C	45 to 150	−2
D	150 to 450	−2
E	450 to 1500	0
F	1500 to 4500	0
G	> 4500	+2

(ii) The mean daily and annual temperature cycles are collected. The recommended temperature differentials between top and bottom of CC slabs of different thickness at various States and regions in India, for the determination of warping stresses are given in Table 7.4.

Table 7.4 Recommended Temperature Differentials in various Regions of the Country

Zone	State and Regions	Temperature differential to °C in slabs of thickness				
		10 cm	15 cm	20 cm	25 cm	30 cm
I	Punjab, U.P., Rajasthan, Gujrat, Haryana, North M.P., excluding hilly regions and coastal areas.	10.2	12.5	13.1	14.3	15.8
II	Bihar, W. Bengal, Assam and E. Orissa excluding hilly and coastal regions.	14.4	15.6	16.4	16.6	16.8
III	Maharastra, Karnataka, South M.P., A.P., W. Orissa and North T.N. excluding hilly and coastal regions.	14.7	17.3	19.0	20.3	21.0
IV	Kerala, South T.N. excluding hilly and coastal regions.	13.2	15.0	16.4	17.6	18.1
V	Coastal areas bounded by hills	12.8	14.6	15.8	16.2	17.0
VI	Coastal areas unbounded by hills	13.6	15.5	17.0	19.0	19.2

(iii) The modulus of subgrade reaction K is determined using standard plate of 75 cm diameter at 0.125 cm deflection. If 30 cm diameter plate is used, the K-value obtained at 0.125 cm deflection is multiplied by 0.5 in order to estimate the K-value of standard plate diameter. The minimum K-value of 5.5 kg/cm² is specified for laying cement concrete pavement. If the K value is lower, a suitable sub-base course may be provided to increase the K-value.

(iv) The flexural strength of cement concrete used in the pavement should not be less than 40 kg/cm². As a general guidance the minimum compressive strength on 15 cm cubes may be taken as 280 kg/cm² at 28 days and mix design strength of 315 to 350 kg/cm², depending upon the degree of quality control. The modulus of elasticity, E and Poisson's ratio, μ may be determined experimentally. The suggested E-value is 3×10^5 kg/cm² and $\mu = 0.15$. The coefficient of thermal expansion of concrete may be taken as 10×10^{-6} per °C for design purposes.

(b) Calculation of Stresses

(i) The wheel load stresses at edge region is calculated for the designed slab thickness as per Westergaard's analysis modified by Teller and Sutherland, (Eq. 7.21), using stress chart given in Fig. 7.23.

(ii) Temperature stress at edge region is calculated as per Westergaard's analysis using Bradbury's coefficient given in Fig. 7.25. Equation 7.24 may be used for the calculation.

(iii) Wheel load stress at corner region is calculated as per Westergaard's analysis modified by Kelley, given in Eq. 7.22 and using the stress chart, Fig. 7.24.

(c) Design Steps for Slab Thickness

(i) The width of slab is decided based on the joint spacings and lane width.

(ii) The length of the CC slab is equal to the spacing of the contraction joints, L_c. This is designed using Eq. 7.30 for plain CC pavements; Eq. 7.31 may be used when reinforcement is provided at the contraction joints for the assumed trial thickness of the slab. However the slab length should conform to the recommendations on *Spacing of Joints* given under Art. 7.4.6 (d).

(iii) A trial thickness value of the slab is assumed for calculating the stresses. The warping stress at edge region is calculated (using Equation 7.26) and this value is subtracted from the allowable flexural stress in concrete to find the residual strength in the pavement to support edge loads.

(iv) The load stress in edge region is found using stress chart, Fig. 7.23 or is calculated (using Eq. 7.21). The available factor of safety in edge load stress with respect to the residual strength is found. If the value of factor of safety is less than 1.0 or is far in excess of 1.0, another trial thickness of the slab is assumed and the calculations are repeated till the factor of safety works out to 1.0 or a slightly higher value for the design thickness *h* cm.

(v) The total stresses at the corner due to wheel load and warping is checked using stress chart Fig. 7.24 and 7.25, (or by using Eq. 7.22 and 7.27) for this thickness *h* cm. If this stress value is less than the allowable, flexural stress in concrete, the slab thickness, h is adequate or else the thickness may be suitably increased (However usually the stresses due to load and warping at the corner region would not be higher than that at edge region).

(vi) The design thickness, *h* is adjusted for the traffic intensity or classification at the end of design life and using the adjustment value from Table to obtain the final adjusted slab thickness.

(d) Spacing of Joints

(i) The maximum spacing recommended for 25 mm wide expansion joints is 140 m when the foundation is rough, for, all slab thicknesses. When the foundation surface is smooth (i.e., surface covered with water proof paper before laying the CC slab) the maximum spacing may be 90 m for slab thicknesses upto 20 cm and spacing of 120 m for slab thickness 25 cm, when the pavement is constructed in summer, however when the pavement is constructed in winter, the above spacings may be restricted to 50 and 60 m respectively.

(ii) The maximum contraction joint spacings may be kept at 4.5 m in unreinforced slabs of all thickness. In the case of reinforced slabs, the contraction joint spacing may be 13 m for 15 cm thick slab with steel reinforcement of 2.7 kg/m^2 and 14 m spacing for 20 cm thick slabs with steel reinforcement of 3.8 kg/m^2.

(e) Design of Dowel Bars

The dowel bar system may be designed on the basis of Bradbury's analysis for load transfer capacity of a single dowel bar in shear, bending and bearing in concrete, using Eq. 7.32, 7.33 and 7.34.

The minimum dowel length is taken as $(L_d + \delta)$, the value of L_d is determined using Eq. 7.35. The load capacity of the dowel system is assumed to be 40% of the design wheel load. The distance on either side of the load position upto which the dowel bars are effective in load transfer is taken as 1.8 times the radius of relative stiffness, *l*.

Dowel bars do not function satisfactorily in thin slabs and therefore dowel bars are provided in slab of thickness 15 cm or more. IRC recommends 2.5 cm diameter dowel bars of length 50 cm to be spaced at 20 cm in the case of 15 cm thick slabs and spaced at 30 cm in the case of 20 cm thick slabs, the design load being 5100 kg.

(f) Design of Tie Bars

Tie bars are designed for longitudinal joints as explained in Art. 7.4.5. Permissible bond stress in deformed bars is 24.6 kg/cm^2 and that in plain tie bars is 17.5 kg/cm^2. Allowable working stress in tensile steel is taken as 1400 kg/cm^2. Recommended details of tie bars are given in Table 7.5 :

Table 7.5

Slab thickness, cm	Tie bar details, cm			
	Diameter	Max. spacing	Plain bars	Deformed bars
15	0.8	38	40	30
	1.0	60	45	35
20	1.0	45	45	35
	1.2	64	55	40
25	1.0	30	45	35
	1.2	55	55	40
	1.4	62	65	46

(g) Design of Reinforcement

Reinforcement in CC pavements are intended to prevent deterioration of the cracks and not to increase the flexural strength of uncracked slabs. The area of longitudinal and transverse steel required per metre width or length of slab is computed from the formula :

$$A = \frac{L f w}{2S} \tag{7.40}$$

where A = area of steel required per m width or length of the slab, cm^2

L = distance between free transverse joints (for longitudinal steel) or free longitudinal joints (for transverse steel), m

f = coefficient of friction between pavement and subgrade, usually taken as 1.5.

S = allowable working stress in steel, usually taken as 1400 kg/cm^2 or 50 to 60 percent of minimum yield stress, kg/cm^2

w = weight of unit area of pavement slab, kg/m^2

The reinforcement may be placed 5 cm below the surface of the slab and is continued across dummy groove joints to serve the purpose of tie bars. At all full depth joints and edges, the reinforcement is kept atleast 5 cm away from the face of joint or edge.

Example 7.24

Design the following details of a plain cement concrete pavement for a two lane highway.

(a) Spacing of expansion and contraction joints

(b) Pavement slab thickness

(c) Dowel bars for expansion joints

(d) Tie bars for longitudinal joints

Follow the design procedure recommended by IRC where ever applicable. Use the given data, IRC load stress charts for edge and corner regions, and assume any other data not provided here.

Width of expansion joint gap = 2.5 cm

Maximum variation in temperature between summer and winter = 35 °C

Thermal coefficient of concrete = 10×10^{-6} per °C

Allowable tensile stress in CC during curing = 0.8 kg/cm^2

Coefficient of friction = 1.5

Unit weight of CC = 2400 kg/cm^3

Design wheel load = 5100 kg

Radius of contact area = 15 cm

Present traffic intensity = 950 commercial vehicles/day

Modulus of reaction of sub-base course = 8 kg/cm^3

Flexural strength (allowable flexural stress) of concrete = 40 kg/cm^2

E value of concrete = 3×10^5 kg/cm^2

μ value = 0.15

Design load transfer through dowel system = 40%

Permissible flexural stress in dowel bar = 1400 kg/cm^2

Permissible shear stress in dowel bar = 1000 kg/cm^2

Permissible bearing stress in concrete = 100 kg/cm^2

Permissible tensile stress in steel (tie bar) = 1400 kg/cm^2

Permissible bond stress in deformed tie bars = 24.6 kg/cm^2

Temperature differential values in the region :

Slab thickness, cm	15	20	25
Temperature differential in slab in the region, °C	14.6	15.8	16.3

Solution

(a) Joint Spacing

$$\delta' = \frac{1}{2}\text{ joint} = \frac{2.5}{2} = 1.25 \text{ cm}$$

$$\text{Spacing of expansion joint } L_s = \frac{\delta'}{100\,C\,(T_2 - T_1)} = \frac{1.25}{100 \times 10 \times 10^{-6} \times 35} = 35.7 \text{ m}$$

which is less than maximum specified spacing of 140 m and so acceptable. Contraction joint spacing in plain CC,

$$L_c = \frac{2 S_c \times 10^4}{W.f} = \frac{2 \times 0.8 \times 10^4}{2400 \times 1.5} = 4.45 \text{ m}$$

which is less than maximum specified spacing of 4.5 m and hence acceptable.

Therefore provide contraction joints at 4.45 m spacing and expansion joints at every 8th such joints i.e., 4.45 × 8 = 35.5 m spacing (instead of 35.7 m).

(b) Pavement Slab Thickness

Assume trial thickness of slab = 20 cm

$$\text{Radius of relative stiffness, } l = \left[\frac{E h^3}{12 K (1-\mu^2)}\right]^{\frac{1}{4}}$$

$$= \left[\frac{3 \times 10^5 \times 20^3}{12 \times 8 (1 - 0.15^2)}\right]^{\frac{1}{4}} = 71.1 \text{ cm}$$

$$\frac{L_x}{l} = \frac{445}{71.1} = 6.26$$

$$\frac{L_y}{l} = \frac{350}{71.1} = 4.92$$

From Fig. 7.25 warping stress coefficient C_x at $\frac{L_x}{l}$ of 6.26 = 0.92.

At $L_y/l = 4.92$, $C_y = 0.72 < C_x$

Temperature differential for 20 cm thick slab = 15.8°C

$$\text{Warping stress at edge, } S_{te} = \frac{C_x . E . e . t}{2}$$

$$= \frac{0.92 \times 3 \times 10^5 \times 10 \times 10^{-6} \times 15.8}{2} = 21.8 \text{ kg/cm}^2$$

Residual strength in concrete slab at edge region

$$= 40.0 - 21.8 = 18.2 \text{ kg/cm}^2$$

Load stress in edge region, using IRC stress chart (Fig. 7.23), corresponding to

$$h = 20, K = 8, S_e = 27.5 \text{ kg/cm}^2$$

$$\text{Factor of safety available} = \frac{\text{residual strength}}{\text{edge load stress}} = \frac{18.2}{27.5} = 0.66$$

As the factor of safety is less than 1.0, it is unsafe. Therefore assume a higher slab thickness say h = 24 cm.

$$l = \left[\frac{3 \times 10^5 \times 24^3}{12 \times 8 (1 - 0.15^2)}\right]^{0.25} = 81.53 \text{ cm}$$

$$L_x/l = \frac{445}{81.53} = 5.46$$

C_x = 0.80 (from chart Fig. 7.25); C_y at L_y/l of 4.29 = 0.6

Temperature differential for 24 cm thick slab (by interpolation) = 16.2°C

$$S_{te} = \frac{1}{2} \times 3 \times 10^5 \times 10 \times 10^{-6} \times 16.2 \times 0.8 = 19.44 \text{ kg/cm}^2$$

Residual strength at the edge

$$= 40.0 - 19.44 = 20.56 \text{ kg/cm}^2$$

Load stress at edge, using stress chart (Fig. 7.23) for

$$h = 4, K = 8, S_e = 19.2 \text{ kg/cm}^2$$

$$\text{Factor of safety available} = \frac{20.56}{19.2} = 1.07 \text{ which is safe and acceptable value}$$

Therefore provide a tentative design thickness of 24 cm.

Check for corner load stress : Using IRC stress chart Fig. 7.24, for h = 24, K = 8, the value of S_c = 23.0 kg per cm^2.

$$\text{Corner warping stress } S_{tc} = \frac{E.e.t}{3(1-\mu)}\sqrt{\frac{a}{l}}$$

$$= \frac{3 \times 15^5 \times 10 \times 10^{-6} \times 16.2}{3(1 - 0.15)}\sqrt{\frac{15}{81.53}} = 7.1 \text{ kg/cm}^2$$

The worst combination of stresses at the corner is 23.0 + 7.1 = 30.1 kg/cm^2, which is also less than the allowable flexural strength of 40 kg/cm^2 and hence the design is safe.

Adjustment for Traffic intensity

$A_d = P' [(1 + r)]^{(n+20)}$

Assuming a growth factor r = 7.5% and the number of years after the last count before the new pavement is opened to traffic, n = 3.

$$A_d = 950\left[1 + \frac{7.5}{100}\right]^{(n+20)} = 5013 \text{ cv/day}$$

This traffic intensity being in the range > 4500, falls in group G and the adjustment factor is + 2 cm.

Therefore the revised design thickness of the slab

$$= 24 + 2 = 26 \text{ cm}$$

(c) *Dowel bars*

Assume dowel bar diameter = 2.5 cm

Joint width, δ = 2.5 cm

For Equal capacity in bending and bearing

$$L_d = 5d\left[\frac{F_t}{F_b} \times \frac{(L_d + 1.5\delta)}{(L_d + 8.8\delta)}\right]^{\frac{1}{2}}$$

$$= 5 \times 2.5\left[\frac{1400}{100} \times \frac{L_d + 1.5 \times 2.5}{L_d + 8.8 \times 2.5}\right]^{\frac{1}{2}}$$

By substituting different values of L_d by trials (as in Example 7.22), the value of L_d is found to be 42.2 cm.

Length of dowel bar = $L_d + \delta$ = 42.2 + 2.5 = 44.7 cm

Therefore provide 45 cm long dowel bars of diameter 2.5 cm

Actual value of L_d = 45.0 − 2.5 = 42.5 cm

Load transfer capacity of single dowel :

$$P'(\text{shear}) = 0.785\, d^2 F_s$$

$$= 0.785 \times 2.5^2 \times 1000 = 4906 \text{ kg}$$

$$P'(\text{bending}) = \frac{2d^2 F_t}{L_d + 8.8\delta} = \frac{2 \times 2.5^5 \times 1400}{42.5 + 8.8 \times 2.5} = 678 \text{ kg}$$

$$P'(\text{bearing}) = \frac{F_b . L_d^2 . d}{12.5(L_d + 1.5\delta)} = \frac{100 \times 42.5^5 \times 2.5}{12.5(42.5 + 1.5 \times 2.5)} = 781 \text{ kg}$$

Taking the lowest value for design, P' (design) = 678 kg

Load capacity factor required :

Load capacity of the dowel group = $5100 \times \frac{40}{100} = 2040$ kg

Capacity factor required = $\frac{2040}{678} = 3.0$

Spacing of dowel bars :

Radius of relative stiffness for revised slab thickness of 24 cm,

$$l = \left[\frac{3 \times 10^5 \times 26^3}{12 \times 8(1 - 0.15^2)}\right]^{1/4} = 86.6 \text{ cm}$$

Effective distance upto which there is load transfer

$$= 1.8\, l = 1.8 \times 86.6 = 155.9 \text{ cm}$$

Assuming a trial spacing of 35 cm between the dowel bars, the capacity available for the group

$$= 1 + \frac{155.9 - 35}{155.9} + \frac{155.9 - 70}{155.9} + \frac{155.9 - 105}{155.9} + \frac{155.9 - 140}{155.9}$$

$= 2.77 <$ the required value of 3.0.

Assume dowel bar spacing of 30 cm.

$$\text{Capacity factor} = 1 + \frac{155.9 - 30}{155.9} + \frac{155.9 - 60}{155.9} + \frac{155.9 - 90}{155.9} + \frac{155.9 - 120}{155.9} + \frac{155.9 - 150}{155.9} = 3.11$$

As this value is greater than the required capacity factor of 3.0, 30 cm spacing of the dowel bars is adequate. Therefore provide 2.5 cm dia. Dowel bars at expansion joints, of total length 45 cm at a spacing of 30 cm centres.

(d) Tie Bars

Area of steel per metre length longitudinal joint,

$$A_s = \frac{b.f.h.W}{100\,S_s} = \frac{3.5 \times 1.5 \times 26 \times 2400}{100 \times 1400} = 2.34 \text{ cm}^2 \text{ per m length}$$

Assuming 1 cm diameter of the bars, cross sectional area of each tie bar $a_s = 0.785 \text{cm}^2$.

Perimeter of the tie bar = 3.14 cm

Number of tie bars required per meter length of joint

$$= \frac{A_s}{a_s} = \frac{2.34}{0.785} = 2.98$$

Spacing of tie bar = $\frac{100}{2.98} = 33.5$ cm

Provide a spacing of tie bar, say 33 cm

Length of plain tie bar, $L_t = \frac{d.S_s}{2S_b} = \frac{1 \times 1400}{2 \times 24.6} = 28.5$ cm

The length of tie bar may be increased by 5 cm for tolerance in placement.

Therefore provide 1 cm diameter deformed tie bars, 34 cm in length at a spacing of 33 cm.

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PROBLEMS

1. Explain 'Flexible and Rigid' pavements and bring out the points of difference.
2. Draw a sketch of flexible pavement cross section and show the component parts. Enumerate the functions and importance of each component of the pavement.
3. What are the various factors to be considered in pavement design ? Discuss the significance of each.
4. Discuss the importance of gross wheel load and contact pressure in stress distribution pattern and in pavement design.
5. Explain ESWL and the concept in the determination of the equivalent wheel load.
6. The loaded weight on the rear dual wheels of a truck is 5500 kg. The centre to centre spacing and clear space in the dual wheels are 30 cm and 10 cm respectively. Calculate the ESWL for pavement thickness (i) 20 cm, (ii) 40 cm, (iii) 70 cm.
7. Discuss the effects of repeated applications of loads on pavements. Explain equivalent wheel load factors for load repetitions.
8. Calculate the design repetitions for ten year period equivalent to 2268 kg wheel load if the mixed traffic in both directions is 1860 vehicles per day. The details of distribution of different wheel loads of commercial vehicles are given below :

Wheel load, kg	Percentage in total traffic volume
2268	25
2722	12
3175	9
3629	6
4082	4
4536	2
4990	1

9. Explain how the elastic modulii of subgrade and base course are estimated using plate bearing test data.
10. Explain how climatic variation affects pavement design and performance.
11. What do you understand by frost action ? Discuss the effects and factors on which the intensity of frost action depends. Suggest measures to prevent or reduce the adverse effects.
12. Enumerate the various methods of flexible pavement design. Briefly indicate the basis of design in each case.
13. Explain group index method of pavement design. What are the limitations of this method ?

14. A subgrade soil sample has the following properties :

    Soil passing soil 0.075 mm sieve = 60%

    Liquid Limit = 55%

    Plastic Limit = 45%

    Design the pavement section by G.I. method for heavy traffic with over 400 commercial vehicles per day.
15. Explain the CBR method of pavement design. How is this method useful to determine thickness of component layers ?
16. Discuss the advantages and limitations of C.B.R. method of design.
17. The CBR value of subgrade soil is 8 percent. Calculate the total thickness of flexible pavement using (i) design curve developed by California State Highway Department (ii) design chart recommended by I.R.C. (iii) design formula developed by the U.S. Corps of Engineers.

    Assume light traffic or 3175 kg wheel load and tyre pressure of 5 kg/cm^2.
18. The C.B.R. test carried out on a subgrade soil gave the following readings :

Penetration, mm	Load, kg	Penetration, mm	Load, kg
0.0	0.0	3.0	58.0
0.5	4.0	4.0	70.0
1.0	14.0	5.0	77.5
1.5	30.0	7.5	93.2
2.0	41.0	10.0	102.5
2.5	50.0	12.5	110.8

The different pavement materials available near the construction site are as follows.

(i) Sandy soil with CBR value = 10%

(ii) Soil-kankar mix with CBR value = 25%

(iii) Broken stone and gravel with CBR = 90%

(iv) Bituminous concrete for surfacing = min. 5 cm thick

Design the pavement structure for commercial vehicles of 2000 per day, with 8.0% growth rate.

19. Explain the California resistance value method of flexible pavement design.
20. Calculate the traffic index value for 10 year period using the following data.

No. of axles	ADT (both directions)
2	700
3	300
4	100
5	20

Assume 60 percent increase in traffic volume in 10 years.

21. Calculate equivalent cohesiometer value for the three pavement component layers given below :

(i) Bituminous concrete surface course 6 cm thick, C-value = 60

(ii) Soil cement base course 15 cm thick, C-value = 250

(iii) Aggregate sub-base course 20 cm thick, C-value = 15

22. Design a flexible pavement, given R-value of subgrade soil = 32, Traffic index = 11.5, C-value of WBM base course = 20 and C-value of 7.5 cm thick bituminous surfacing = 65.

23. Design pavement thickness by California 'R' value method using the following data :

(i) Traffic index =10.5

(ii) C-value of base course = 15

(iii) C-value of 8.0 cm thick bituminous surface course = 60

(iv) Test results on subgrade soil are as under :

Moisture content, percent	R-value	Pressure, kg/cm^2	
		Expansion	Exudation
16.5	60	1.2	46.5
19.0	41	0.9	82.0
22.5	15	0.4	20.0

24. Briefly explain the flexible pavement design method using triaxial test results.

25. Calculate thickness of bituminous mat using triaxial method.

E-value of subgrade soil	=	90 kg/cm^2
E-value of paving materials	=	900 kg/cm^2
Wheel load	=	5100 kg
Tyre pressure	=	7 kg/cm^2
Traffic coefficient	=	1.25
Saturation coefficient	=	0.8

26. Design the thickness of pavement layers in problem no. 25, above, if base course and sub-base course are to be provided having E-values of 400 and 200 kg/cm^2.

27. Explain McLeod method of pavement design.

28. A plate bearing test using 30 cm diameter plate carried out on a subgrade which yielded a pressure of 3 kg/cm^2 after 10 load repetitions at 0.5 cm deflection. Design a highway pavement for a wheel load of 5100 kg with a tyre pressure of 7 kg/cm^2.

29. Discuss the application of Burmister's two layer theory in pavement design.

30. Plate bearing test conducted with 30 cm diameter plate on a subgrade sustained a load of 1500 kg at 0.25 cm deflection. The test when carried out on a base course of thickness 18 cm sustained a load of 5500 kg at 0.25 cm deflection. Design the pavement section for a wheel load of 5500 kg with tyre pressure of 7.5 kg/cm^2 using Burmister's approach.

31. What are the considerations for design of rigid pavements ?

32. Explain the following terms :

(a) modulus of subgrade reaction

(b) radius of relative stiffness

(c) radius of resisting section

33. Find the radius of relative stiffness and radius of resisting section for a concrete slab from the following data :

Modulus of elasticity of concrete	=	3.1×10^5 kg/cm^2
Poisson's ratio of concrete	=	0.15
Modulus of subgrade reaction	=	6.0 kg/cm^2
Thickness of concrete slab	=	22 cm
Radius of loaded area	=	16 cm

34. Explain the critical locations of loading as regards wheel load stresses in cement concrete pavement. Discuss the Westergaard's concept and assumptions.

35. What are the advantages of Bradbury's stress coefficients and other load stress charts ?

36. Calculate the stresses at interior, edge and corner of a cement concrete pavement by:

(a) Westergaards's stress equations

(b) IRC stress charts

Modulus of elasticity of concrete	= 3.0×10^5 kg/cm^2
Poisson's ratio of concrete	= 0.15
Thickness of concrete pavement	= 18 cm
Modulus of subgrade reaction	= 8.5 kg/cm^2
Wheel load	= 5100 kg
Radius of loaded area	= 15 cm

37. Estimate the thickness of cement concrete pavement using the method suggested by Indian Roads Congress.

Modulus of elasticity of concrete	= 3.0×10^5 kg/cm^2
Modulus of rupture of concrete	= 40 kg/cm^2
Poisson's ratio of concrete	= 0.15
Modulus of subgrade reaction	= 6 kg/cm^2
Wheel load	= 5100 kg
Radius of contact area	= 15 cm

38. Explain the design considerations for spacing of :

(a) Expansion joints.

(b) Contraction joints with and without reinforcement.

39. Calculate the spacing of expansion joint from the following data :

Maximum joint width = 2 cm

Temperature of laying concrete = 20°C

Maximum slab temperature expected = 55°C

Coefficient of thermal expansion = 10×10^{-6} per °C

40. Find the spacing between contraction joints for a 3.5 m slab width having a thickness of 22 cm for

(a) Plain concrete slab

(b) R.C.C. slab

The allowable tensile stress values in concrete and steel are 0.8 and 1400 kg/m^2, coefficient of friction is 1.5.

41. Discuss the design details of dowel bars.

42. Explain how the dimensions and spacing of tie bars are designed.

43. A cement concrete pavement 20 cm thick and 7.5 m width has a longitudinal joint along the centre line. Design the diameter, length and spacing of the tie bars, if the allowable working stress in steel is 1400 kg/cm^2 in tension, allowable bond strength of deformed bars in concrete is 24.6 kg per cm^2 and coefficient of friction is 1.2. Assume unit weight of concrete as 2400 kg/cm^3.

44. Discuss Westergaard's concept of temperature stresses in concrete pavements.

45. Calculate the warping stresses at interior, edge and corner for a concrete pavement of thickness 20 cm with transverse joints at 4.5 m spacing. The width of slab is 3.5 m. For concrete E = 3×10^5 kg/cm^2 and $\mu = 0.15$, K value for subgrade = 5 kg/cm^3. Temperature differential is 0.9 C per cm. Assume thermal coefficient for concrete as 10×10^{-6} per°C.

46. Explain the effect due to the expansion and contraction of cement concrete slab and discuss the types of stresses induced.

47. Discuss the critical combination of stresses due to wheel load and temperature effects.

48. Briefly outline the I.R.C. recommendations for determining the thickness of cement concrete pavement.

49. Design the CC pavement thickness, expansion and contraction joint spacing, dowel add tie bars for a wheel load of 5100 kg. Assume all data suitably.

Chapter 8
Highway Construction

8.1 GENERAL CONSTRUCTION

8.1.1 Introduction

In article 1.5, it was stated that the science of highway engineering raises some fundamental questions as to what is a road or highway, how is it planned and designed and lastly how is it built. By now in the preceding chapters, the treatment of the subject has answered the questions pertaining to *planning, geometrics, materials and structural design* of highways. In this chapter, it is attempted to discuss in detail the question how are the highways built i.e. highway construction.

Engineers have been always with open mind to adopt any material available to them for its use for the construction purposes. Research facilities at hand help them to judge the suitable of the materials. It is logical to see that the purpose of highway construction is to provide a firm and even surface for the carriageway or the pavement which could stand the stress caused due to number of load applications. Hence the pavement is prepared using broken stones, or soil (for stabilized soil) and binder material to bind the aggregates to form a homogenous pavement section. The binders employed are mainly soil slurry, bituminous material and cement.

Depending upon the desired strength of the pavement, the aggregate gradations and the type and proportion of binders are decided. These three basic binder medium give rise to a number of construction methods.

8.1.2 Types of Highway Construction

The highway types are classified as below :

(i) Earth road and gravel roads

(ii) Soil stabilized roads

(iii) Water bound macadam (WBM) road

(iv) Bituminous or black-top roads

(v) Cement concrete roads

The roads in India are classified based on location and functions. All the roads do not cater for the same amount of traffic volume or intensity. Since the funds available at hand for financing the construction projects are also meager, it is necessary to have roads which cost less. The adoption of low cost roads is now preferred in developing countries like India where large lengths of roads are to be constructed in the rural areas with the limited finances available in the country. Earth roads and stabilized roads are typical examples of low cost roads. Stabilised soil roads are gaining importance in the form of *low cost* roads. The details of the soil stabilization techniques and construction methods are given in Chapter 9.

Water bound macadam construction which uses the crushed stones is the main technique adopted in India since *eighteenth century*. Even the present day roads are by and large constructed on the existing water bound macadam bases. Here the binding is done through moist soil or stone dust. The binder being a slurry, its inherent drawbacks can be foreseen. The abrasion and attrition phenomena, the formation of dust during dry weather and mud during monsoons makes it necessary to provide a superior type of surface layer to protect it. This brings the use of bituminous binder. Thus most of the roads with bituminous surface have water bound macadam base. The function of the bituminous binder in black top roads is mainly to provide a thin bituminous film coating over the aggregates. When this mixture is compacted together, the bituminous film provides binding or adhesion between the aggregate pieces. The phenomenon is purely physical. The use of bitumen binder with the variation in mode of its application, the aggregate gradation, and the mode and degree of compaction have resulted in different techniques of bituminous pavement construction. The bituminous binder being a semi-viscous material, the resulting pavement does not possess the rigidity comparable with the cement concrete pavement.

In the cement concrete pavement, Portland cement is used with sand and aggregates to form a cement concrete mix. This type of pavement falls in a group of *rigid* pavement and is considered as a high type of pavement. This pavement type suits any climatic or traffic condition and can be built on almost all soil types. Reinforcement is some times provided to construct a reinforced concrete pavement. Prestressed concrete pavements are also gaining importance in some cases.

Pavement Types

The selection of base course and the surface course depends upon the following factors :

(i) Type and intensity of traffic.

(ii) Funds available for the construction project and for the subsequent maintenance.

(iii) Subgrade soil and drainage conditions.

(iv) Availability of construction materials at site.

(v) Climatic condition.

(vi) Plants and equipment available.

(vii) Time available for completing the project.

(viii) Altitude at which construction has to be done.

The selection of the pavement type therefore is governed by one or more factors listed above. However the fundamental factor is the traffic volume or intensity. It is suggested that the low cost roads could successfully function with traffic intensity of 30 tonnes to

200 tonnes per day. The water bound macadam may cater for over 500 tonnes per day. The bituminous pavements and cement concrete roads are considered suitable for heavier traffic intensity. This however would depend upon the thickness of pavement section, type of soil subgrade and the desired service life of the road. Each type of pavement has different periods of life after which it starts deteriorating.

In highway construction usually *stage construction* technique is adopted. This means improving the pavement structure in stages to match with increase in road traffic. This becomes all the more imperative for developing countries like India, because of their financial handicaps with regard to the initial investment for the high type of pavement construction like that of cement concrete at the first instance.

The highway construction project may be broadly divided in two phases :

(i) Earthwork and preparation of subgrade.

(ii) Pavement structure.

The earth work mainly consists of preparation of the subgrade, to be suitable to receive the subsequent construction of pavement structure.

8.2 EARTHWORK

8.2.1 General

The subgrade soil is prepared by bringing is to the desired grade and camber and by compacting adequately. The subgrade may be either in embankment or in excavation, depending on the topography and the finalized vertical alignment of the road to be constructed.

The earth work quantities are estimated based on longitudinal and transverse section along the alignment of the road. In order to reduce the cost of construction, it is necessary to plan the movement of materials from cuts to the nearest fills. It is necessary to decide the limits of *economical haul* and *lift*. It is advantageous to plot a *mass haul diagram* to compute the haulage details. The swelling and shrinkage factor may also be considered in the excavation and compaction of earth.

8.2.2 Excavation

Excavation is the process of cutting or loosening and removing earth including rock form its original position, transporting and dumping it as a fill or spoil bank. The excavation or cutting may be needed in soil, soft rock or even in hard rock, before preparing the subgrade. The selection of excavation equipment and the cost analysis is made based on the stiffness of the materials to be excavated.

Earth excavation work may be divided as excavation or cutting, grading and compaction. The depth of the excavation is decided, among other factors, on requirement of vertical profile of the road. The slope to be provided is governed by the type of soil including stratification, if any, and the depth of the cutting. The stability computations may help in arriving at the maximum permissible slope for the complex problems. However highway cuts much flatter slopes are preferred from other considerations including aesthetics. Construction of side drains also require excavations along road side.

Excavation equipment

The excavation equipment commonly used in highway projects include bull dozers, scrapers, power shovels, draglines, clamshells and hoes. However, in small projects excavation is carried out manually using hand tools.

Bull dozer and scraper may be used for shallow excavation work and for hauling the earth for relatively short distances. *Bull dozer* is considered to be a versatile machine for many construction projects as it may be used for clearing site, opening up pilot roads, moving earth for short haul distances of about 100 m and also in several other jobs. *Scraper* is considered as one of the useful earth-moving equipment as it is self operating; it can dig, haul and discharge the material in uniformly thick layers. However scrapers are not capable of digging very stiff material.

Power shovel is used primarily to excavate earth of all classes except rock and to load it into wagons. Power shovels may be mounted on crawler tracks and so they can move at low speeds. Figure 8.1 shows the operation and basic parts of a power shovel; these include the mounting, cab, boom, dipper stick, dipper and hoist line. The power shovel can effectively operate to excavate earth from a lower level where it stands and when the depth of the face to be excavated is not too shallow.

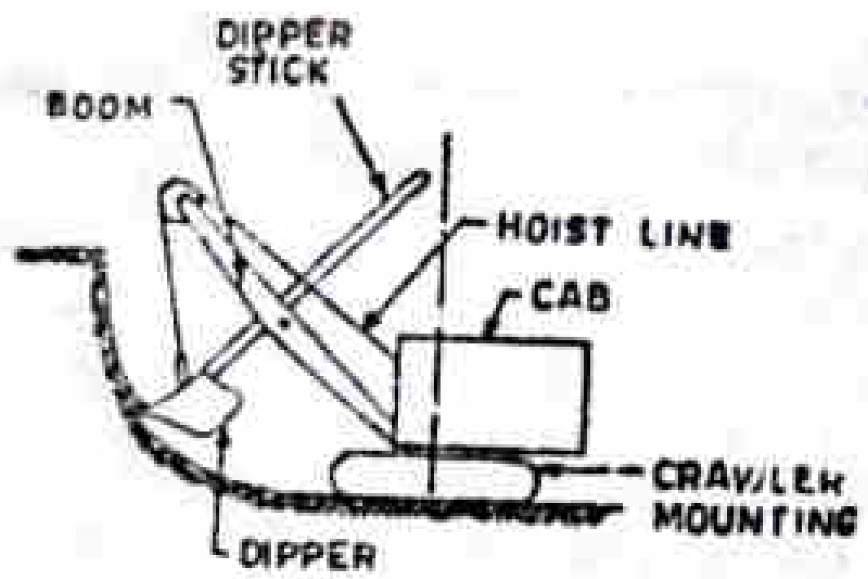


Fig. 8.1 Power Shovel

Dragline is used to excavated soft earth and to deposit in nearby banks or to load into wagons. Dragline may also be mounted on crawler. It can operate on natural ground while excavating from a pit with the bucket; thus it is not necessary for the dragline to go into the pit in order to excavate. The basic parts and operation of a dragline are shown in Fig. 8.2. The bucket is thrown out from the dragline on the top of the earth to be excavated and then pulled back towards the base of the machine.

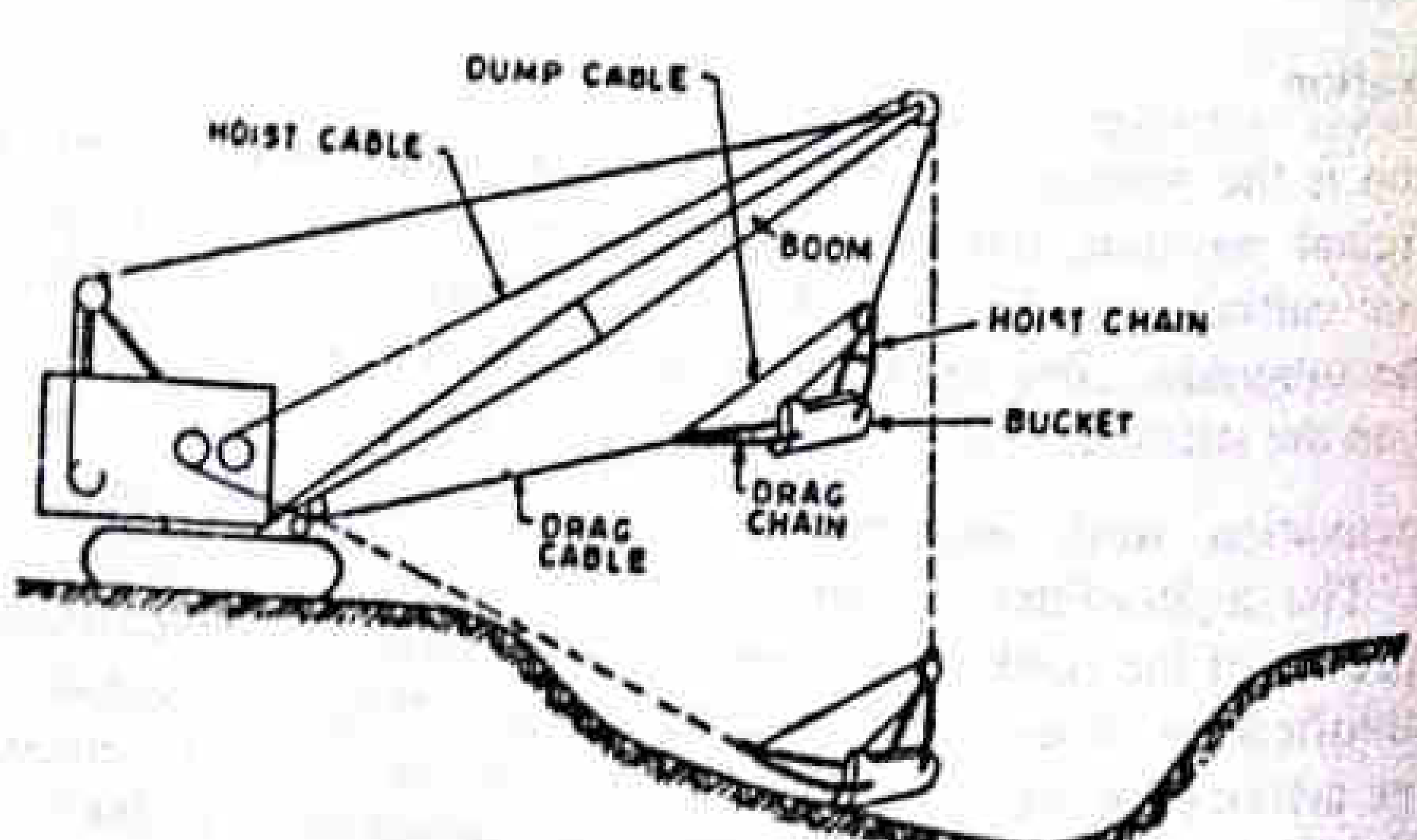


Fig. 8.2 Dragline

Clam shell consists of a bucket of two halves or *shell* which are hinged together at top. The shells may be attached to the shovel-crane units or at the boom of a drag line. The open clam-shell bucket is thrown on the top of the loose material to be dug and as the

bucket is lifted, the two halves close entrapping the material into the bucket. Figure 8.3 shows the clamshell bucket. This equipment is useful for excavation of soft to medium materials and loose material at or below existing ground surface.

Hoe is an excavating equipment of the power-shovel family. Hoe is meant to excavate below the natural surface where the machine is stationed and is capable of having precise control of depth of excavation at close range work (see Fig. 8.4). Hoe can exert high tooth pressures and hence can excavate stiff material which normally can not be excavated by dragline.

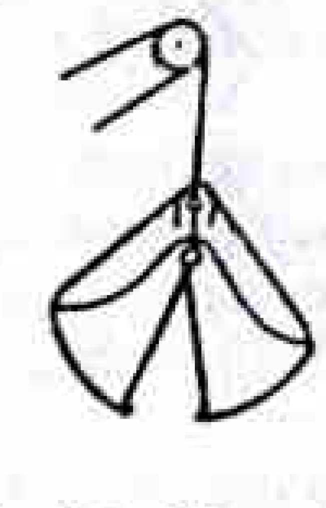


Fig. 8.3 Clamshell

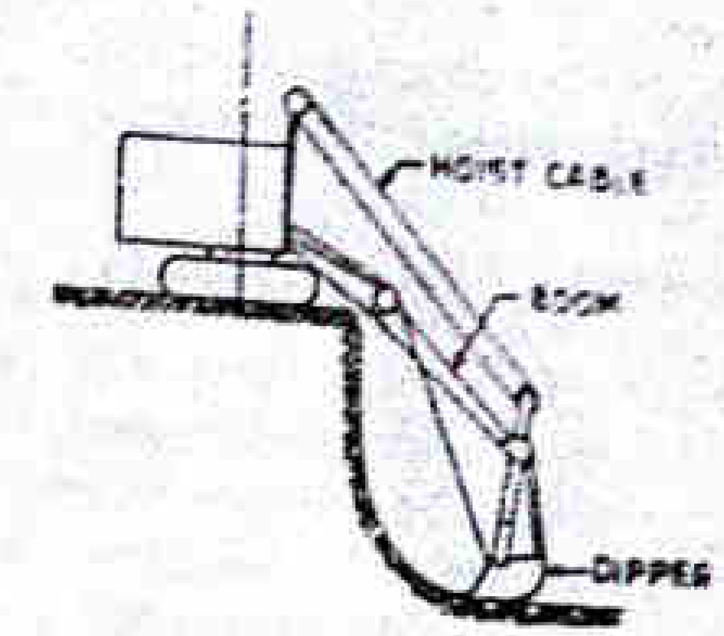


Fig. 8.4 Hoe

The excavation equipment is selected depending on the nature of the material, the distance, to be hauled and the method of disposal. At times the selection may be made based on the availability also.

8.2.3 Embankment

When it is required to raise the grade line of a highway above the existing ground level it becomes necessary to construct embankments. The grade line may be raised due to any of the following reasons :

(i) to keep the subgrade above the high ground water table.

(ii) to prevent damage to pavement due to surface water and capillary water.

(iii) to maintain the design standards of the highway with respect to the vertical alignment.

The design elements in highway embankments are :

(i) height

(ii) fill material

(iii) settlement

(iv) stability of foundation, and

(v) stability of slopes

Height

The height of the embankment depends on the desired grade line of the highway and the soil profile or topography. Also the height of the fill is some times governed by stability of foundation, particularly when the foundation soil is weak.

Fill Material

Granular soil is generally preferred as highway embankment material. Silts, and clays are considered less desirable. Organic soils, particularly peat are unsuitable. The best of the soils available locally is often selected with a view to keep the lead and lift as low as possible. At times light-weight fill material like *cinder* may be used to reduce the weight when foundation soil is weak.

Settlement

The embankment may settle after the completion of construction either due to consolidation and settlement of the foundation or due to settlement of the fill or due to both. If the embankment foundation consists of compressible soil with high moisture content, the consolidation can occur due to increase in the load. The settlement of the fill is generally due to inadequate compaction during construction and hence by proper compaction this type of settlement may be almost eliminated. Whatever be the type of settlement, it is desirable that the settlement is almost complete before the construction of pavement. To accelerate the rate of consolidation of saturated foundation clay, *vertical sand drains* are sometimes constructed. These are vertical columns of sand installed in the compressible foundation like marshy soils in order to decrease drainage path and thus accelerate the rate of consolidation. The vertical sand columns may be of 30 to 60 cm diameter and 2.5 to 6 meter spacing, arranged in a hexagonal pattern. A horizontal sand blanket, 40 to 60 cm thick is placed at the top of the drains extending across the entire width of embankment at its bottom. This helps the water to flow out with ease. Figure 8.5 shows a typical cross section of embankment with vertical sand drains.

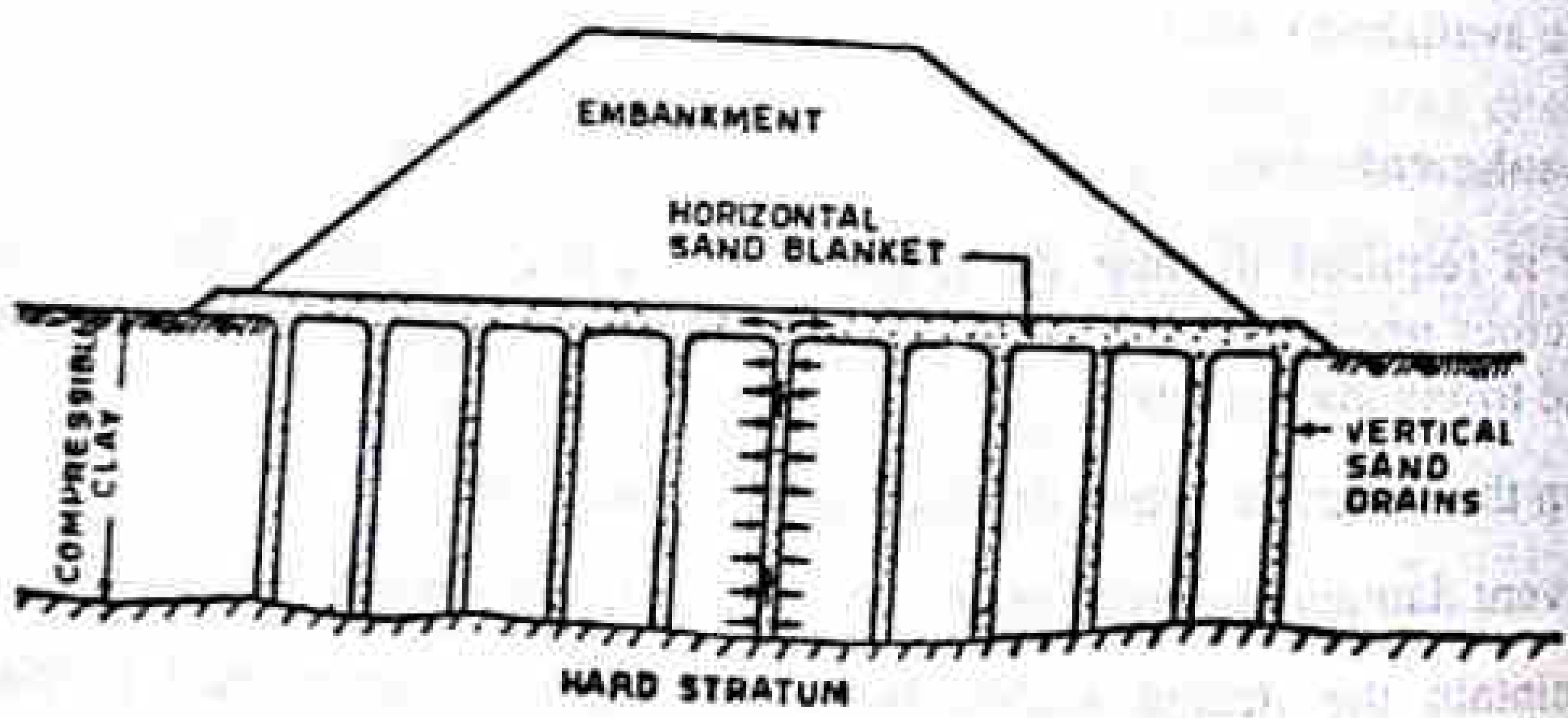


Fig. 8.5 Embankment with Vertical Sand Drains at Foundation

Stability of Foundation

When the embankment foundation consists of weak soil just beneath or at a certain depth below in the form of a weak stratum, it is essential to consider the stability of the foundation against a failure. This is all the more essential in the case of high embankments. The foundation stability is evaluated and the factor of safety is estimated by any of the following approaches :

(a) assuming a certain failure surface such as a circular arc or any other composite shape and analysing it with *Swedish circular arc* analysis or method of *wedges*, as the case may be.

(b) estimating the average shear stress and strength at the foundation layers by approximate methods and estimating the factor of safety.

(c) using theoretical analysis based on elastic theory.

The factor of safety in the case of compressible soil foundation is likely to be minimum just after the completion of the embankment. Later due to consolidation of foundation and consequent gain in strength there will be an increase in the foundation factor of safety. Thus it is evident that in such compressible foundation soils, the vertical sand drains would be useful also to increase the rate of gain in strength. By proper design of vertical sand drains, it is possible to limit the decrease in foundation factor of safety due to the construction, within the allowable value.

Stability of Slopes

The embankment slopes should be stable enough to eliminate the possibility of a failure under adverse moisture and other conditions. Hence the stability of the slope should be checked or the slope should be designed providing minimum factor of safety of 1.5. Often much flatter slopes are preferred in highway embankments due to aesthetic and other reasons.

Construction of embankments

The embankment may be constructed either by rolling in relatively thin layers or by hydraulic fills. The former is called *rolled-earth method* and is preferred in highway embankments. Each layer is compacted by rolling to a satisfactory degree or to a desired density before the next layer is placed. Compaction is carried out at optimum moisture content so as to take advantages of maximum dry density using a specified compacting effort and equipment. The thickness of the layers may vary between 10 and 30 cm depending on various factors such as soils type, equipment, specifications etc.

The practice of dumping the earth without compacting properly and allowing the fill to get consolidated under weather during few subsequent season should be avoided as the settlement will continue for a very long period. If pavement is constructed before the settlement of the fill is almost complete, the pavement is likely to become uneven and also fail later-on. Compaction of soil is discussed separately in Art. 8.2.5.

8.2.4 Preparation of Subgrade

The preparation of subgrade includes all operations before the pavement structure could be laid over it and compacted. Thus the preparation of subgrade would include site clearance, grading (embankment or cut section) and compaction.

The subgrade may be situated on embankment or excavation or at the existing ground surface. In all the cases, site should be cleared off and the top soil consisting of grass, roots rubbish and other organic matter are to be removed. Next, the *grading operation* is started so as to bring the vertical profile of the subgrade to designed grade and camber. Bull dozers, scrapers and blade graders are useful equipment to speed up this work. It is most essential to compact the top of subgrade, upto a depth of about adequately before placing the pavement layer.

8.2.5 Soil Compaction

By compaction of soil, the particles are mechanically constrained to be packed more closely, by expelling part of the air voids. Compaction increases the density and stability, reduces settlement and lowers the adverse effects of moisture. Hence proper compaction of fills, subgrade, sub-base and base course are considered essential for proper highway construction.

The various factors influencing soil compaction include the moisture content, amount and type of compaction, soil type and stone content. It is a well known fact that there is an *optimum moisture content* (OMC) for a soil which would give *maximum dry density* for a particular type and amount of compaction. Hence it is always desirable to compact the soil at the OMC after deciding the compacting equipment.

Compacting equipment

Soil compaction is achieved in the field either by rolling, ramming or by vibration. Hence the compacting equipment may also be classified as rollers, rammers and vibrators. Compaction of sands are also achieved by watering ponding and jetting.

Rollers

The principle of rollers is the application of pressure, which is slowly increased and then decreased. The various type of rollers which are used for compaction are smooth wheel, pneumatic tyred and sheepsfoot rollers. Further the construction equipment such as trucks, tractors and bull dozers also help in compaction of the materials to some extent.

Smooth Wheeled Rollers

There are two types of smooth wheeled rollers, one three-wheeled or macadam rollers, and the other tandem rollers. The gross weight of the former type range between 4 to 18 tonnes whereas that of the latter type with two axles varies between 1 to 14 tonnes. The compacting efficiency of the smooth wheeled roller depends on the weight, width and diameter of each roller. The smooth wheeled rollers are suitable to roll a wide range of soils, preferably granular soils and pavement materials for the various layers. These are particularly found to be useful in compacting soils and other materials where a crushing action is advantageous.

Pneumatic Tyred Roller

In this type number of pneumatic wheels are mounted on two or more axles, under a loading platform. These rollers are pulled by tractors. The pneumatic tyred rollers are considered to be most suitable to compact nonplastic silts and fine sands. In addition to the direct pressure due to rolling, there is also a slight kneading action. The mechanics of compaction by pneumatic tyred rollers is illustrated in Fig. 8.6.

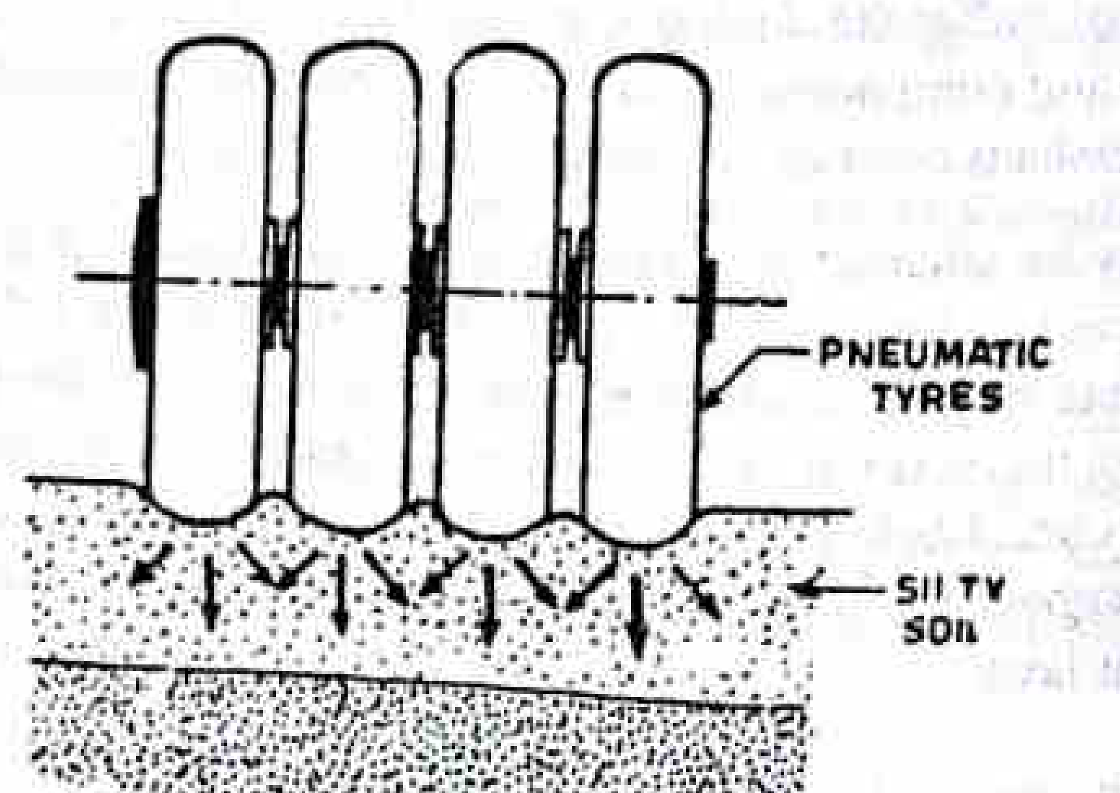


Fig. 8.6 Compaction by Pneumatic Tyred Roller

Sheepsfoot Roller

This type of roller consists of hollow steel cylinder with projecting feet. The weight of the roller can be increased by filling the drum with wet soil. The weight, diameter and width of the roller may be varied and also the shape and size of the feet. These may be pulled by tractors. The efficiency of the sheepsfoot rollers depends on the weight of the roller and the number of feet in contact with the ground at a time. Sheepsfoot rollers are considered more suited to compact clayey soils. During rolling operations the soil under the projecting feet get compacted and also there is a considerable kneading action to the soil. The thickness of compacting layer is kept about 5 cm more than the length of each foot. Figure 8.7 illustrates the mechanics of compaction under a sheepsfoot roller. About 24 or more number of passes of the roller may be necessary to obtain adequate compaction. However the top layer of the subgrade or fill may be compacted using smooth wheeled roller so as to get a properly finished surface.

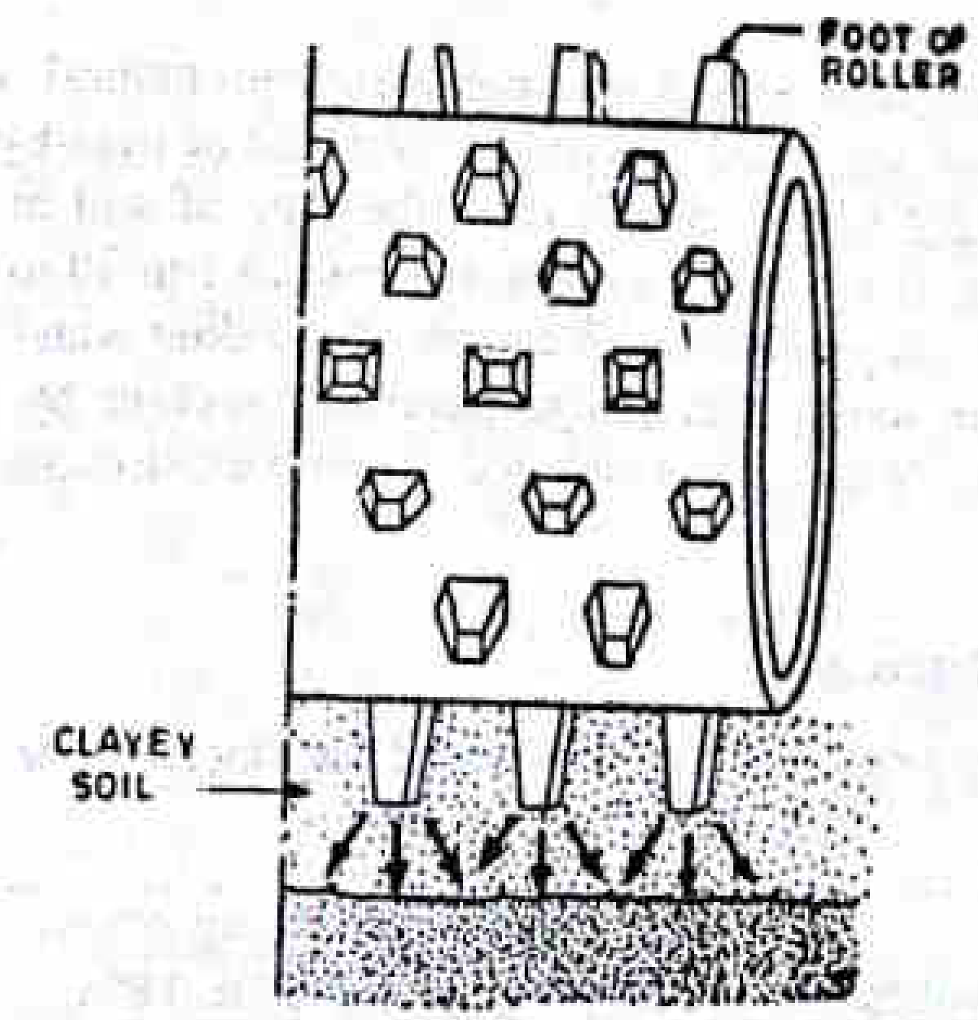


Fig. 8.7 Compaction by Sheepsfoot Roller

Rammers are useful to compact relatively small areas and where the roller cannot operate such as compaction of trenches, foundation and slopes. The output rammer is much lower than that of roller.

Vibrators are most suited for compacting dry cohesionless granular material. There are also vibrator mounted roller to give the combined effects of rolling and vibration. Vibratory rollers are advantageously used in compacting a wide range of materials.

Watering (jetting and ponding) is considered to be an efficient method of compacting cohesionless sands. Watering heavily and rolling by smooth wheel of pneumatic tyred roller may also give adequate compaction of cohesionless sands.

Field Control for Compaction

For adequate quality control in construction, it is essential to have proper field control in compaction. The two field control tests needed are :

(i) Measurement of moisture content

(ii) Measurement of dry density

The moisture content of the soil may be found before compaction by any one of the rapid methods suitable at the site. If the moisture is controlled at the OMC, then the next control needed is the dry density, the desired value of which may be achieved by increasing the number of passes for the selected equipment and the thickness of each layer. Dry density may be found by any suitable method; the *sand replacement method* is considered quite satisfactory.

A certain percentage (say 100 or 95 %) of the *standard density* is generally aimed at in the field compaction. Thus by field checks it is possible to control the construction to achieve adequate compaction. However, statistical quality control methods should be followed for the compaction in construction of high embankments.

8.3 CONSTRUCTION OF EARTH ROADS

8.3.1 General

An earth road is the cheapest type of road prepared from natural soil. The pavement section is totally made out of the soil available at site and at near-by borrow pits. The type of construction by and large, depends upon the type of soil at site. The camber provided to the earth roads is very steep and ranges between 1 in 20 to 1 in 33. The steep cross slope helps to keep the pavement surface free of standing water; otherwise the soil being pervious, the water would damage the pavement section by softening it. The maximum cross slopes of 1 in 20 is recommended to avoid erosion due to rain waters and formation of cross ruts.

8.3.2 Specification of Materials

Soils of the following properties are considered satisfactory for constructing earth roads :

	Base course	Wearing courses
Clay content	< 5%	10 to 18%
Silt content	9 to 32%	5 to 15%
Sand content	60 to 80%	65 to 80%
Liquid limit	< 35%	< 35%
Plasticity index	< 6%	4 to 10%

8.3.3 Construction Procedure

The construction of earth road may be divided into the following steps :

(i) *Material.* The soil survey is carried out and suitable borrow pits are located within economical haulage distances. The borrow pits are usually selected outside the land width. The trees, shrubs, grass roots and other organic matter including top soil are removed before excavating earth for construction.

(ii) *Location.* The centre line and road edges are marked on the ground along the alignment, by driving wooden pegs. *Reference pegs* are also driven to help in following the desired vertical profile of the road during construction. The spacing of the reference pegs depends on the estimated length of road construction per day.

(iii) *Preparation of subgrade.* The various operations involved in the preparation of subgrade are as follows :

(a) Clearing site

(b) Excavating and construction of fills to bring the road to a desired grade.

(c) Shaping of subgrade.

The site clearance may be carried out manually using appliances like spade, pick and hand shovel. Mechanical equipment like dozer, scraper and ripper may also be used for the purpose.

Construction of fills and excavation of costs to bring the road profile to the desired grade may also be done either manually or using excavation, hauling and compaction equipment. The details have been given in Art. 8.2.

Dozers are however considered very useful for short hauling distances. If the work is carried out manually, more time is required. The compaction of fill if manually done, will not be proper, it should be left to get consolidated under atmospheric condition. The equipment used by manual labour include hand shovel, spade, pick-axe, baskets, rammers and hand rollers.

The subgrade should be graded to the desired camber and longitudinal profile and to the desired depth depending on the thickness of pavement construction. It is desirable to compact the subgrade before placing the pavement layers.

(iv) *Pavement construction.* The borrowed soil (more than one soil type mixed to the desired proportion, if necessary) is dumped on the prepared subgrade and pulverized. The field moisture content is checked and additional water is added, if necessary, to bring it upto OMC. The soil is mixed, spread and roller in layers such that the compacted thickness of each layer does not exceed 10 cm. The type of roller for compaction is decided based on soil type, desired amount of compaction and availability of equipment. Also see Art. 8.2.5. Atleast 95 percent of dry density of I.S. light compaction is considered desirable. The camber of the finished pavement surface is checked and corrected if necessary.

(v) *Opening to traffic.* The compacted earth road is allowed to dry out for a few days before opening to traffic.

8.4 CONSTRUCTION OF GRAVEL ROADS

8.4.1 General

Gravel roads are considered superior to earth roads as they can carry heavier traffic. The road consists of a carriageway constructed using the gravels. The camber may be between 1 in 25 and 1 in 30.

A well compacted crushed rock or gravel road is fairly resilient and does not become slippery when wet. This type of road can cater for about 100 tonnes of pneumatic tyred vehicle or 60 tonnes of iron tyred vehicles per day per lane. Two types of construction methods are generally followed. They are the *feather edge type* and the *trench type* as shown in Fig. 8.8. The feather edge type is constructed over the subgrade with varying thickness, so as to obtain the desired cross slope for the pavement surface. In the trench type, the subgrade is prepared by excavating a shallow trench. Since there is better confinement for the gravel, the trench type is preferred.

8.4.2 Material

Hard variety of crushed stone or gravel of specified gradation is used. However softer varieties of stone may also be utilised. There are no specifications for the materials. Rounded stones and river gravel are not preferable as there is poor interlocking.

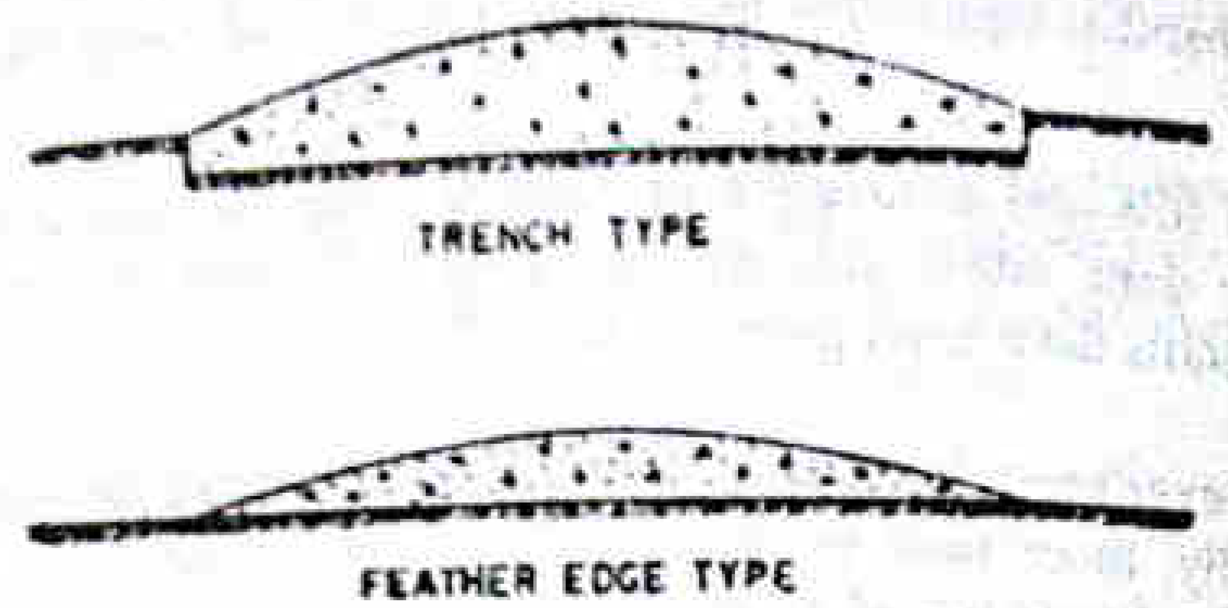


Fig. 8.8 Types of Gravel Road Sections

8.4.3 Construction Procedure

(i) *Material* Gravel to be used for the construction is stacked along the sides of the proposed road.

(ii) *Location* Pegs are driven as explained under article 8.3.

(iii) *Preparation of subgrade.* Site is cleared and fills and cuts are completed. Trench is formed to the desired depth of construction, the width of the trench is made equal to that of the carriageway. The trench is brought to the desired grade and is compacted.

(iv) *Pavement construction.* Crushed gravel aggregates are placed carefully in the trench so as to avoid seggregation. Aggregates are spread with greater thickness at centre and less towards the edges so as to obtain the desired camber. The layer is rolled using smooth wheeled rollers starting from the edges and proceeding towards the centre with an overlap of atleast half the width of roller in the longitudinal direction. Some quantity of water may also be sprayed and rolling is done again so that the compaction is effective. The camber is checked and corrected from time to time using a template or camber board.

(v) *Opening to traffic.* A few days after the final rolling and drying out, the road is opened to the traffic.

8.5 CONSTRUCTION OF WATER BOUND MACADAM ROADS

8.5.1 General

The water bound macadam (WBM) is the construction known after the name of *John Macadam*. (See also article 2.16 and 2.17). The term macadam in the present day means, the pavement base course made of crushed or broken aggregate mechanically interlocked by rolling and the voids filled with screening and binding material with the assistance of water. The WBM may be used as a sub-base, base course or surfacing course. The thickness of each compacted layer of WBM ranges from 10.0 cm to 7.5 cm depending on the size and gradation of the aggregates used. The number of layers and total thickness of WBM construction depends on the design details of the pavement.

When used as a surfacing course, WBM gets deteriorated rapidly under adverse conditions of traffic and weather; therefore it is desirable to provide a bituminous surfacing course over the WBM layer in order to prolong its life.

8.5.2 Specifications of Materials for WBM Pavement

Type of course Aggregates

The course aggregates used in WBM generally consists of hard varieties of crushed aggregates or broken stones. However, soft aggregates like over burnt bricks metal or naturally occurring soft aggregates such as kankar or laterite may be used. Crushed slag obtained from blast furnace may also be used. The weaker varieties of aggregates which get crushed during rolling need not strictly conform to the requirements of gradation and there is no need of using screenings for filling the voids in the coarse aggregates.

Property	Requirements for pavement layer		
	Sub-base	Base course	Surfacing course
(i) Los Angeles abrasion value (maximum value, percent) or	60	50	40
(ii) Aggregate impact value (maximum value, percent)	50	40	30
(iii) Flakiness index (maximum value, percent)	-	15	15

Properties of Coarse Aggregates

The crushed stone aggregate should be generally hard, durable and of acceptable shape, free from flaky and elongated particles. The IRC specifies the following physical requirements of coarse aggregates for WBM construction, in terms of the test value for the three pavement layers.

Size and Grading Requirements of Coarse Aggregates

The coarse aggregates for each layer of construction should, as far as possible conform to any one of the three gradings specified below. Grading No. 1 consists of coarse aggregates of size range 90 to 40 mm and is more suitable for sub-base course. Thickness of compacted layer is usually 100 mm. Grading No. 2 consists of aggregates of size range 63 to 40 mm and grading No. 3 of range 50 to 20 mm and compacted thickness of each layer is normally 75 mm.

Grading No.	Size range, mm	Sieve size, mm	Percent passing the sieve, by weight
1	90 to 40	100	100
		80	65 – 85
		63	25 – 60
		40	0 – 15
		20	0 – 5
2	63 to 40	80	100
		63	90 – 100
		50	30 – 70
		40	0 – 15
		20	0 – 5
3	50 to 20	63	100
		50	95 – 100
		40	35 – 70
		20	0 – 10
		10	0 – 5

Screenings

The screenings are used to fill up the voids in the compacted layer of coarse aggregates. The screenings consist of aggregates of smaller size, generally of the same material as the coarse aggregates. The grading requirements of screenings for WBM construction are given below :

Classification grading	Size of screenings mm	Sieve size, mm	Percent passing the sieve, by weight
A	12.5	12.5	100
		10.0	90 – 100
		4.75	10 – 30
		0.15	0 – 8
B	10.0	10.0	100
		4.75	85 – 100
		0.15	10 – 30

The IRC has suggested that from economic considerations, predominantly non-plastic materials such as kankar nodules, moorum or gravel (other than river borne rounded aggregate) may be utilised as screening material, provided the liquid limit of the material is less than 20%, plasticity index is less than 6.0% and the portion of fines passing 0.075 mm sieve is less than 10%. However, if crushable type of materials are used as coarse aggregate, the use of screenings may be dispensed with.

Binding Material

Binding material consisting of fine grained material is used in WBM construction to prevent raveling of the stones. Kankar nodules or lime stone dust may also be utilised, if locally available. The binding material with plasticity index value 4 to 9% is used in WBM surface course construction; the plasticity index of binding course material should be less than 6.0% in the case of WBM layers used as base course or sub-base course, with bituminous surfacing. If the screenings used consist of crushable material like moorum or soft gravel, there is no need to apply binding material, unless the plasticity index value is low.

Quantity of Materials

(i) The approximate loose quantities of materials required in m^3 for 10 cm compacted thickness of WBM sub-base using coarse aggregate of grading no 1 per 10 m^2 area are :

(a) Coarse aggregate size 90 to 40 mm = 1.21 to 1.43

(b) Stone screening type A, 12.5 mm size = 0.40 to 0.44

or

crushable type screenings (moorum/gravel) = 0.44 to 0.47

(c) Binding material for sub-base course = 0.88 to 0.10

(ii) The approximate loose quantities of materials required in m^3 for 7.5 cm compacted thickness of WBM base course or surfacing course using coarse aggregate of grading No. 2 per 10 m^2 area are :

(a) Coarse aggregate size 63 to 40 mm = 0.91 to 1.07

(b) Stone screening type A, 12.5 mm size for base course = 0.18 to 0.21

Stone screenings for surfacing course = 0.15 to 0.17

Alternatively, Stone screenings type B,

9.0 mm size for base course = 0.30 to 0.33

9.0 mm size for surfacing = 0.24 to 0.26

Alternatively, crushable type screenings = 0.33 to 0.35

(c) Binding material for base course = 0.06 to 0.09

Binding material for surfacing course = 0.10 to 0.15

(Note : Binding material is not required if crushable type of screening is used).

(iii) The approximate loose quantities of materials required in m^3 for 7.5 cm compacted thickness of WBM base or surfacing course using coarse aggregate of grading No. 3 per 10 m^2 area are :

(a) Coarse aggregate size 50 to 20 mm = 0.91 to 1.07

(b) Stone screenings type B, 9.0 mm size for base = 0.27 to 0.30

Stone screenings type B, 9.0 mm size for surfacing = 0.22 to 0.24

Alternatively crushable type screenings = 0.33 to 0.35

(c) Binding material for base course = 0.06 to 0.09

Binding material for surfacing course = 0.10 to 0.15

(Note : Binding material is not required if crushable type screening is used.)

8.5.3 Construction Procedure

Preparation of Foundation for Receiving the WBM Course

The foundation for receiving the new layer of WBM may be either the subgrade or sub-base or base course. This foundation layer is prepared to the required grade and camber and the dust and either loose materials are cleaned. On existing road surfaces, the depressions and pot-holes are filled and the corrugations are removed by scarifying and reshaping the surface to the required grade and camber as necessary. If the existing surface is a bituminous surfacing, furrows of depth 50 mm and width 50 mm are cut at 1.0 m intervals and at 45 degrees to the centre line of the carriageway before laying the coarse aggregate.

Provision of Lateral Confinement

Lateral confinement is to be provided before starting WBM construction. This may be done by constructing the shoulders to advance, to a thickness equal to that of the compacted WBM layer and by trimming the inner sides vertically.

Spreading of Coarse Aggregates

The coarse aggregates are spread uniformly to proper profile to even thickness upon the prepared foundation and checked by templates. The WBM course is normally constructed to compacted thickness of 7.5 cm except in the case of WBM sub-base course using coarse aggregate grading no.1 which is of 10.0 cm compacted thickness.

Rolling

After spreading the coarse aggregates properly, compaction is done by a three wheeled power roller of capacity 6 to 10 tonnes or alternatively by an equivalent vibratory roller, the weight of the roller depends on the type of coarse aggregates.

Rolling is started from the edges, the roller being run forward and backward until the edges are compacted. The run of the roller is then gradually shifted towards the centre line of the road, uniformly overlapping each preceding rear wheel track by one half width. This process is repeated by rolling from either edge towards the centre line until adequate compaction is achieved.

On superelevated portions of the road, rolling is commenced from the inner or lower edge and progressed gradually towards the outer or upper edge of the pavement.

Application of Screenings

After the coarse aggregates are rolled adequately, the dry screenings are applied gradually over the surface to fill the interstices in three or more applications. Dry rolling is continued as the screenings are being spread and brooming carried out.

Sprinkling and Grouting

After the application of screenings, the surface is sprinkled with water, swept and rolled. Wet screenings are swept into the voids using hand brooms. Additional screenings are applied and rolled till the coarse aggregates are well bonded and firmly set.

Application of Binding Material

After the application of screening and rolling, binding material is applied at a uniform and slow rate at two or more successive thin layers. After each application of binding material, the surface is copiously sprinkled with water and wet slurry swept with brooms to fill the voids. This is followed by rolling with a 6 to 10 tonnes roller and water is applied to the wheels to wash down the binding materials that sticks to the roller. When crushable type screenings like moorum or gravel are used, there is no need to apply binding materials, except in the surfacing course.

Setting and Drying

After final compaction, the WBM course is allowed to set over-night. On the next day the 'hungry' spots are located and are filled with screenings or binding material, lightly sprinkled with water if necessary and rolled. No traffic is allowed till the WBM layer sets and dries out. In the case of WBM base course, the layer is allowed to dry completely without permitting traffic to ply and then the bituminous surfacing is laid. Limited construction traffic may be permitted to ply over the WBM layer taking proper care not to damage the layer.

8.5.4 Checking of Surface Evenness and Rectification of Defects

The surface evenness of longitudinal direction is checked by 3.0 m straight edge and the number of undulations exceeding 12 mm in the case of WBM layer of grading no. 1 and 10 mm in the case of grading nos. 2 and 3 are recorded in each completed length of 300 m; the maximum number of undulations permitted in each case is 30. The spots with 15 mm undulations are marked for rectification of defects.

The cross profile is checked using camber template and the maximum variation from special profile should not exceed 12 mm in the case of aggregate grading no.1 and 8 mm in the case of gradings 2 and 3.

Where the unevenness exceed the specified limits, the defective area with a minimum area of 10 m^2 at a spot is scarified, reshaped with added material or fresh material and re-compacted properly as specified earlier.

8.6 CONSTRUCTION OF BITUMINOUS PAVEMENTS

8.6.1 Introduction

Bituminous pavements are in common use in India and abroad. It is possible to construct relatively thin bituminous pavement layers over an existing pavement. Therefore, these are commonly adopted as wearing course. Flexible pavement could be strengthened in stages by constructing bituminous pavement layers one after another in a certain period of time unlike the cement concrete pavement construction. There are a wide range of construction materials (type, size and grading of aggregates and type and grade of bituminous binder) and bituminous pavement construction techniques in use. Variation in design and construction types have given rise to the bituminous paving technology. It is well realized that the excessive binder content over an optimum value for a given mix is detrimental to the good performance of the black top pavements. This is contrary to the role of cement as a binder in the cement concrete mixes, where the excess of the binder does not decrease the strength. Therefore, based on the surface area of the aggregates, and the technique of construction the optimum binder content may be determined. Another problem associated with the construction of bituminous pavements is control of the proper viscosity of the bituminous-aggregate mixtures during mixing and compaction operations.

In this country, the bituminous construction is by and large adopted on the surface coarse. In fact till recent years the bituminous construction as a wearing course or a surface course was considered as the main treatment. Bituminous constructions are also adopted for base and binder courses of pavements on heavy-traffic roads. Different from the cement concrete surfacing which would require very high cost of construction and a substantial curing period before opening the road to traffic, the bituminous surfacing has a distinct advantage in this respect. The black top construction is in extensive use in developing nations like, India where the cement as a construction material is in great demand for large number of other engineering projects. Also stage development is possible in the case of bituminous roads, depending on traffic demands.

8.6.2 Types of Bituminous Construction

Number of types and methods are in use for bituminous pavement construction. It is attempted to broadly classify them here based on the methods of construction. The following construction techniques are in use :

(i) Interface treatments like prime coat and tack coat

(ii) Surface dressing and seal coat

(iii) Grouted or penetration type constructions -

(a) Penetration Macadam

(b) Built-up Spray Grout

(iv) Premix which may be any of the following :

(a) Bituminous bound macadam

(b) Carpet

(c) Bituminous concrete

(d) Sheet asphalt or rolled asphalt

(e) Mastic asphalt

In the above methods the bituminous binders used are either straight run bitumen, road tar, cutback or emulsion. The choice of particular binder depends upon the type of construction, availability of materials and equipment, climatic conditions etc. Bitumen and tar, require heating to bring them to a proper viscosity for their use. The construction technique using these materials, is termed as *hot mix* technique. Cutback and emulsion do not normally require heating are therefore applied cold and the technique is known as *cold mix*. Thus the construction techniques are classified into two groups, namely, hot mix and cold mix.

The methods of bituminous construction employed in base course are, grouted or penetration macadam (either fullgrout or semi-grout) and bitumen bound macadam. The different bituminous construction used in surface course of pavements are surface dressing, grouted or penetration macadam with seal coat, and bituminous premix surfacing. The premix may be either carpet, bituminous concrete, sheet asphalt or mastic asphalt.

Based on the technique of mixing and construction, the bituminous constructions are classified as *road mix* and *central plant mix* methods.

Depending upon the gradation, the premix constructions are classified as *open graded* and *dense graded*. The premixed carpet is open graded whereas the bituminous concrete is dense graded.

8.6.3 Explanatory Notes on Bituminous Construction Types

(i) Interface Treatment

Thus surface of the existing pavement layer is to be cleaned to remove dust and dirt and a thin layer of bituminous binder is to be sprayed before the construction of any type of bituminous layer over this surface. This treatment with bituminous material is called interface treatment which is necessary to provide the necessary bond between the old and the new layers. The interface treatment may either be a *prime coat* or a *tack coat* and in some cases, the prime coat followed by a tack coat.

(a) Prime coat : Bituminous prime coat is the first application of a low viscosity liquid bituminous material over an existing porous or absorbent pavement surface like the WBM base course. The main object of priming is to plug in the capillary voids of the porous surface and to bond the loose mineral particles on the existing surface, using a binder of low viscosity which can penetrate into the voids. Usually MC or SC cutbacks of suitable grade or viscosity is chosen depending on the porosity of the surface to be treated. The bituminous primer is sprayed uniformly using a mechanical sprayer at a rate of 7.3 to 14.6 kg per 10 m^2 area, depending on the porosity of the surface. The primed surface is allowed to cure for atleast 24 hours, during which period no traffic is allowed.

(b) Tack coat. Bituminous tack coat is the application of bituminous material over an existing pavement surface which is relatively impervious like an existing bituminous

surface or a cement concrete pavement or a pervious surface like the WBM which has already been treated by a prime coat. Tack coat is usually applied by spraying bituminous material of higher viscosity like the hot bitumen at the rate of 4.9 to 9.8 kg per 10 m^2 area depending in the type of the surface. However in some special circumstances, a tack coat of bituminous emulsion may also be applied in cold state.

(ii) Bituminous Surface Dressing

Bituminous Surface Dressing (BSD) is provided over an existing pavement to serve as thin wearing coat. The single coat surface dressing consists of a single application of bituminous binder material followed by spreading of aggregate cover and rolling. The principle of this method is illustrated in Fig. 8.9.

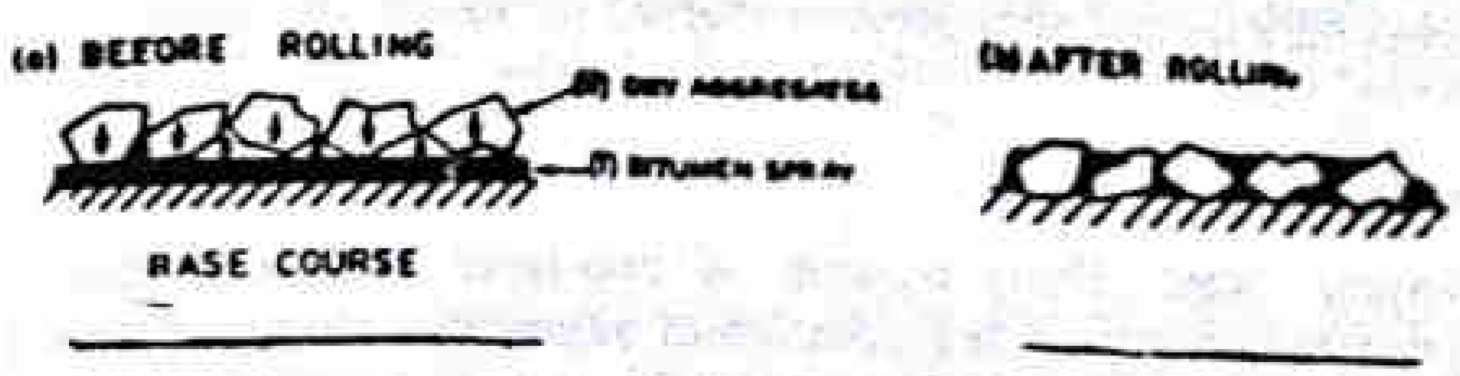


Fig. 8.9 Principle of Bituminous Surface Dressing Construction

When the surface dressing is similarly done in two layers, it is called 'two coat bituminous surface dressing'. Where better adhesion is required, aggregates precoated with bituminous binder are used in lieu of conventional surface dressing and this technique is called 'Bituminous Surface Dressing with Precoated Aggregates'. The main functions of BSD are :

(a) to serve as a thin wearing course of pavement and to protect the base course

(b) to water proof the pavement surface and to prevent infiltration of surface water

(c) to provide dust-free pavement surface in dry weather and mud-free pavement in wet weather.

(iii) Seal Coat

Seal coat is usually recommended as a top coat over certain bituminous pavements which are not impervious, such as open graded bituminous constructions like premixed carpet and grouted Macadam. Seal coat is also provided over an existing bituminous pavement which is worn out. The seal coat is a very thin surface treatment or a single coat surface dressing which is usually applied over an existing black top surface. A premixed sand bitumen (hot mix) seal coat is also commonly used over the premixed carpet. The main functions of seal coat are :

(a) to seal the surfacing against the ingress of water

(b) to develop skid resistant texture

(c) to enliven an existing dry or weathered bituminous surface.

(iv) Penetration Macadam

Bituminous Penetration Macadam or Grouted Macadam is used as a base or binder course. The coarse aggregates are first spread and compacted well in dry state and after that hot bituminous binder of relatively high viscosity is sprayed in fairly large quantity at the top. The bitumen penetrates into the voids from the surface of the compacted aggregates, thus filling up a part of the voids and binding some stone aggregates together. The principle of Penetration Macadam construction is illustrated in Fig. 8.10.

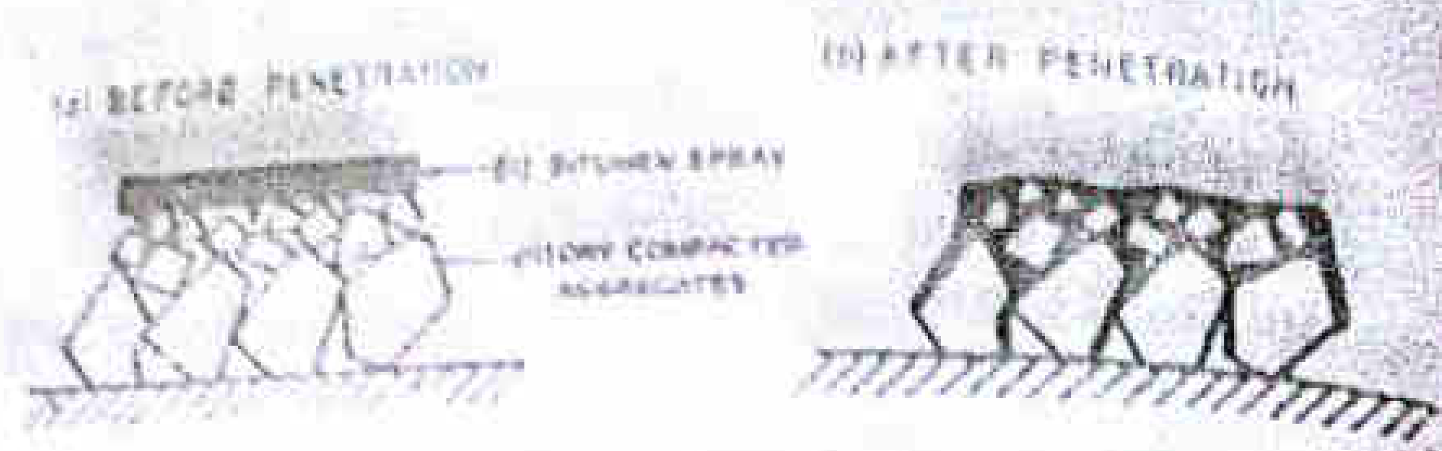


Fig. 8.10 Principle of Bituminous Penetration Macadam Construction

Depending upon the quantity of bitumen spread and the extent of penetration, it is called 'full grout' when bitumen penetrated to full depth of compacted aggregates and 'semi-grout' when it penetrates upto about half the depth. Full grout is adopted in regions of heavy rain fall and semi-grout is adopted in regions of moderate rain fall and traffic.

(v) Built-up Spray Grout

Built-up Spray Grout (BSG) consists of two-layer composite construction of compacted crushed aggregates with application of bituminous binder after each layer for bonding and finished with key aggregates at the top to provide a total compacted thickness of 75 mm. This method is commonly used for strengthening of existing bituminous pavements. A suitable wearing course is invariably provided over this layer before opening to traffic.

(vi) Premix Methods

In this group of methods the aggregates and the bituminous binder are mixed thoroughly before spreading and compacting. It is possible to coat each particle of aggregate with the binder still the quantity of binder used may be considerably lesser than penetration macadam type construction. In premixed constructions, the quantity of bitumen used could be precisely controlled and they offer increased stability of the mix even with lower bitumen contents. Depending on the gradation of the aggregates chosen, premixed constructions maybe classified as open graded; semidense and dense mixes. The common types of premixed bituminous constructions are bituminous macadam, bituminous carpet, and bituminous concrete. Other types of premixed constructions include sheet asphalt and mastic asphalt.

(vii) Bituminous Macadam

Bituminous Macadam (BM) or Bitumen Bound Macadam is a premixed construction method consisting of one or more courses of compacted crushed aggregates premixed with bituminous binder, laid immediately after mixing. The BM is laid in compacted thicknesses of 75 mm or 50 mm and three different gradations of aggregates have been suggested for each thickness to provide open graded and semi-dense constructions. The BM is essentially a base course or binder course and hence should be covered by a suitable surfacing course before exposing to traffic. BM base course is considered to be much superior than other types of base course materials such as WBM with respect to load dispersion characteristics and durability.

(viii) Bituminous Premixed Carpet

Premixed Carpet (PC) consists of coarse aggregates of 12.5 and 10.0 mm sizes, premixed with bitumen or tar binder are compacted to a thickness of 20 mm to serve as a surface course of the pavement. Being a open graded construction, the PC is to be

invariably covered by a suitable seal coat such as premixed sand-bitumen seal coat before opening to traffic. The PC consists of all aggregates passing 25 mm and retained on 6.7 mm sieve. When a fairly well graded material as per specification is used for the construction of the bituminous carpet of thickness 20 to 25 mm, the construction method is called semi-dense carpet.

(ix) Bituminous Concrete or Asphalt Concrete

Bituminous Concrete or Asphaltic Concrete (AC) is a dense graded premixed bituminous mix which is well compacted to form a high quality pavement surface course. The AC consists of a carefully proportioned mixture of coarse aggregates, fine aggregates, mineral filler and bitumen and the mix is designed by an appropriate method such as the Marshall method to fulfil the requirements of stability, density, flexibility and voids. The thickness of bituminous concrete surface course layer usually ranges from 40 to 75 mm. The IRC has provided specification for 40 mm thick AC surface course for highway pavements.

(x) Sheet Asphalt

Sheet asphalt or rolled asphalt is a dense sand-bitumen premix of compacted thickness 25 mm. used as a wearing course. The sheet asphalt consists of well graded coarse to fine sand (without coarse aggregates) and a suitable penetration grade bitumen to form a dense and impervious layer. This is usually laid over cement concrete pavement to provide an excellent riding surface. The sheet asphalt also protects the joints in cement concrete pavements and could cause a reduction in warping stresses due to a decrease in the temperature variations between top and bottom of the concrete slab.

(xi) Mastic Asphalt

Mastic asphalt is a mixture of bitumen, fine aggregates and filler in suitable proportions which yields a voidless and impermeable mass. Though the ingredients in mastic asphalt are similar to those in bituminous concrete, properties of mastic asphalts are quite different. The mastic asphalts when cooled results in a hard, stable and durable layer suitable to withstand heavy traffic. This material also can absorb vibrations and has a property of self-healing of cracks without bleeding. It is a suitable surfacing materials for bridge deck slabs.

The filler, bitumen binder and aggregate are taken in suitable proportion and they are heated in sequence, and cooked at a temperature of 200 to 223°C according to the binder grade, for over 5 hours in a special cooker. At a temperature of about 200°C the mastic asphalt has such a consistency that it can flow. But on cooling to atmospheric temperature, it hardens to semi-solid or solid state. Because of this property even without compaction a voidless hard layer of mastic asphalt can be obtained. The mastic asphalt should be spread at a temperature of about 200°C to a thickness between 2.5 to 5 cm. No rolling is required.

8.6.4 Methods of Construction

Premix types of bituminous constructions are generally carried out in the field using appropriate plants. There are two types of mixing plant and travelling plant.

The centre mixing plant consists of units for batching different materials, separate heating units for mixed aggregates and bitumen and a mixing unit of large capacity. The aggregates, filler and bitumen are transported to the site of the mixing plant which is stationed at a suitable location and the bituminous mix is again transported from the plant to the construction site. Generally there is very good control on the quality of the mix obtained from the central mixing plant.

The travelling plant is a smaller unit and can be shifted from time to time along the road side as the bituminous construction progresses. In hot mix constructions the heated aggregates are mixed with heated bitumen in a central or travelling plant. In cold mix method, the aggregates at atmospheric temperature are mixed with bituminous material of low viscosity in cold state or after slight heating.

8.6.5 Bituminous Construction Procedures

In this article, the material requirements, plant and equipment and construction process employed for the following types of bituminous constructions have been presented :

(i) Surface dressing

(ii) Grouted or penetration macadam

(iii) Built-up spray grout

(iv) Bitumen bound macadam

(v) Bituminous carpet

(vi) Bituminous concrete

Construction Procedure for Surface Dressing

The bituminous surface dressing (BSD) is done either in a single coat or in two coats over the existing bituminous pavement and for renewal or as a wearing course over a water bound macadam road. Also there is a specification for surface dressing using aggregates precoated with bitumen.

The surface dressing work is done only in dry and clear weather when the atmospheric temperature is above 16°C.

Specifications of Materials

The requirement of materials greatly differs depending upon whether the application is in a single coat or two coats and whether the dressing is done on a black top pavement or on a WBM pavement.

The grade of bitumen is selected depending upon the climatic conditions. However bitumen grades between 80/100 and 180/200 are frequently used. Tar or cutback may also be used.

The binder quantity requirement for surface dressing is given in Table 8.1. Coarse aggregates (preferably crushed stone or crushed gravel) should be clean, strong and durable fairly cubical in shape.

Table 8.1 Binder Quantity for Surface Dressing

Base course Type	Binder requirement kg per 10 m^2 area					
	First or Single coat			Second coat		
	Bitumen	Tar	Cutback	Bitumen	Tar	Cutback
(i) WBM	17 to 19.5	17 to 22	19 to 22	10 to 12	12 to 15	12 to 15
(ii) Renewal of black top to surfacing	10 to 12	10 to 17	10 to 12	10 to 12	10 to 17	10 to 12

Aggregate type should conform to the following requirements :

Los Angles abrasion value	35 percent max.
Aggregate impact value	30 percent max.
Flakiness index	25 percent max.
Water absorption	1 percent max.
Stripping at 40°C after 24 hours immersion (CRRI test)	25 percent max.

The recommended size of chippings and quality required are given below :

	Sieve size, mm			Quantity in m^3 for 10 m^2
	Passing	Retained	Normal	
First coat, 12 mm thick	18	9	12	0.14 to 0.15
Second coat or renewal coat, 9 mm thick	12	6	9	0.09 to 0.11

Plants and Equipment

Equipment for heating of bitumen, mechanical sprayer, mechanical blower or hand brushes and roller are required. Some of these are explained in subsequent methods.

Construction Steps

(i) *Preparation of existing surface* : The existing surface is prepared to the proper profile and ruts, depressions etc. are rectified before the treatment is done. The surface is made free of dust or loose materials. A prime coat is applied if the existing base course has a previous surface such as soil stabilized material or WBM.

(ii) *Application of binder* : On a prepared surface using a mechanical sprayer or pouring-can, uniform spraying of the bituminous binder is done at the specified rate. Care is taken that excessive binder is not applied to the localised areas as this would cause *bleeding*.

(iii) *Application of stone chippings* : After the application of binder, the cover material i.e., stone chippings as per the requirement is spread to cover the surface uniformly.

(iv) *Rolling of first or final coat* : The rolling is done with tandem roller of 6 to 8 tonnes weight after the cover material is spread. Rolling is started from the edges proceeding towards the centre longitudinally with overlapping not less than one third of the roller tread. When rolling of one half width upto the centre is completed this way, the rolling is carried out on the other half again starting from the edge and proceeding towards the centre line. This is continued until the particles are firmly inter-locked. This is the final rolling if the surface dressing is in single coat.

If the second coat is applied then the rolling is done again after the treatment is done for second coat.

(v) *Application of binder and stone chippings for second coat* : The binder is again applied to the prepared surface as per requirements. Immediately after this cover material of smaller aggregates is spread as before.

(vi) *Rolling of second coat* : Soon after the application of the materials, the rolling is done as described in (iv) above.

(vii) *Finishing and opening to traffic* : The surface is checked for longitudinal and cross profile using a straight edge of length 3.0 metre and variation in surface greater than 6.0 mm are corrected. The road section is opened to traffic after 24 hours.

Construction Procedure for Penetration (Grouted) Macadam

The construction of a penetration macadam is recommended for thickness of 50 and 75 mm.

Specifications of Materials

IRC recommends to use any grade of bitumen from 80/100, 60/70 and 30/40. Road tars, RT-4 and RT-5 could also be used. The quantity of bitumen required depends on the desired degree of penetration of binder into the compacted aggregate layer.

The physical requirements of stone aggregates are specified by the following test values :

Los Angles abrasion value	40 percent max.
Aggregate impact value	30 percent max.
Flakiness index	25 percent max.
Stripping at 40°C after 24 hours immersion (CRRI test)	25 percent max.
Loss with sodium sulphate, 5 cycles	12 percent max.

The grading of the coarse aggregate and key aggregates as recommended by IRC is given in Table 8.2 and 8.3.

Table 8.2 Gradings of Coarse Aggregates for Penetration macadam

Percent passing sieve size, mm	Compacted thickness , mm	
	50	75
63	-	100
50	100	-
38	-	35-70
25	35-70	-
19	-	0-15
12	0-15	-
9	-	-
4.75	-	-
2.36	0-5	0-5

Table 8.3 Key Aggregates for Penetration Macadam

Percent passing sieve size, mm	Compacted thickness , mm	
	50	75
38	-	-
25	-	100
19	100	35-70
12	35-70	-
9	-	0-15
4.75	0-15	-
2.36	0-15	0-5

The coarse aggregates required for 50 mm compacted thickness is 0.06 m^2 per 10 m^2 area and for 75 mm compacted thickness is 0.90 m^3 per 10 m^3.

The quantity of key aggregates required for 50 and 75 mm compacted thickness are 0.15 and 0.18 m^3 per 10 m^3 area.

Plants and Equipment

Various equipment needed are :

bitumen heating device;	bitumen distributor
aggregate spreader;	roller for compaction

For a small job bitumen container is heated under the direct flame. For a continuous project, the heating is done within the bitumen distributor. See Fig. 8.11.

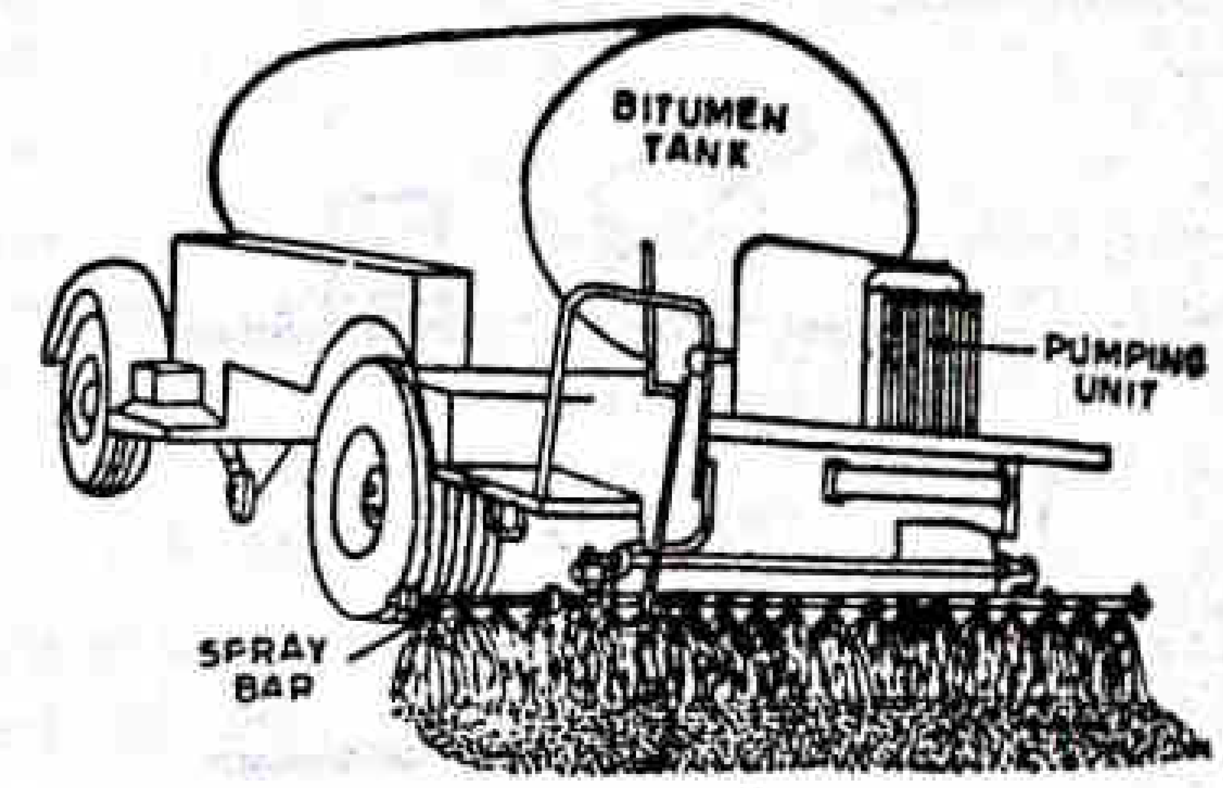


Fig. 8.11 Bitumen Distributor

Different types of aggregate spreaders used are :

- Whirl spreader
- Vane spreader
- Hopper spreader
- Self propelled spreader

These are shown in Fig. 8.12. Two types of rollers used for compaction are tandem rollers and vibratory rollers.

Construction Steps

(i) *Preparation of existing surface* : The underlying course is prepared and conditioned to a uniform grade. The surface is lightly scarified and is brushed. Priming coat may be applied if required on porous surfaces.

(ii) *Spreading the coarse aggregates* : The coarse aggregates are spread with proper edge protection. This is done either by mechanical spreader or by hand. A template cut to camber profile is used o achieve the desired profile in cross section.

(iii) *Rolling* : The aggregates are dry-rolled with 10 tonnes roller until the aggregates are compacted and interlocked. Rolling is commenced from the sides and proceeded to the centre, the overlap recommended being 30 cm. The amount of rolling depends upon the aggregate type. In all cases the rolling is stopped with either first sign of crushing or when the internal movements of the aggregates is stopped. Then dry compacted coarse aggregates are checked for desired profile using 3.0 metre straight edge and corrected where the irregularities exceed 12 mm.

(iv) *Bitumen application* : Over the dry and compacted coarse aggregates, the binder is applied uniformly either with pressure distributor or mechanical hand sprayer. The quantity of bitumen required for this purpose is 50 and 68 kg per 10 m^2 for 50 and 75 mm compacted thickness respectively.

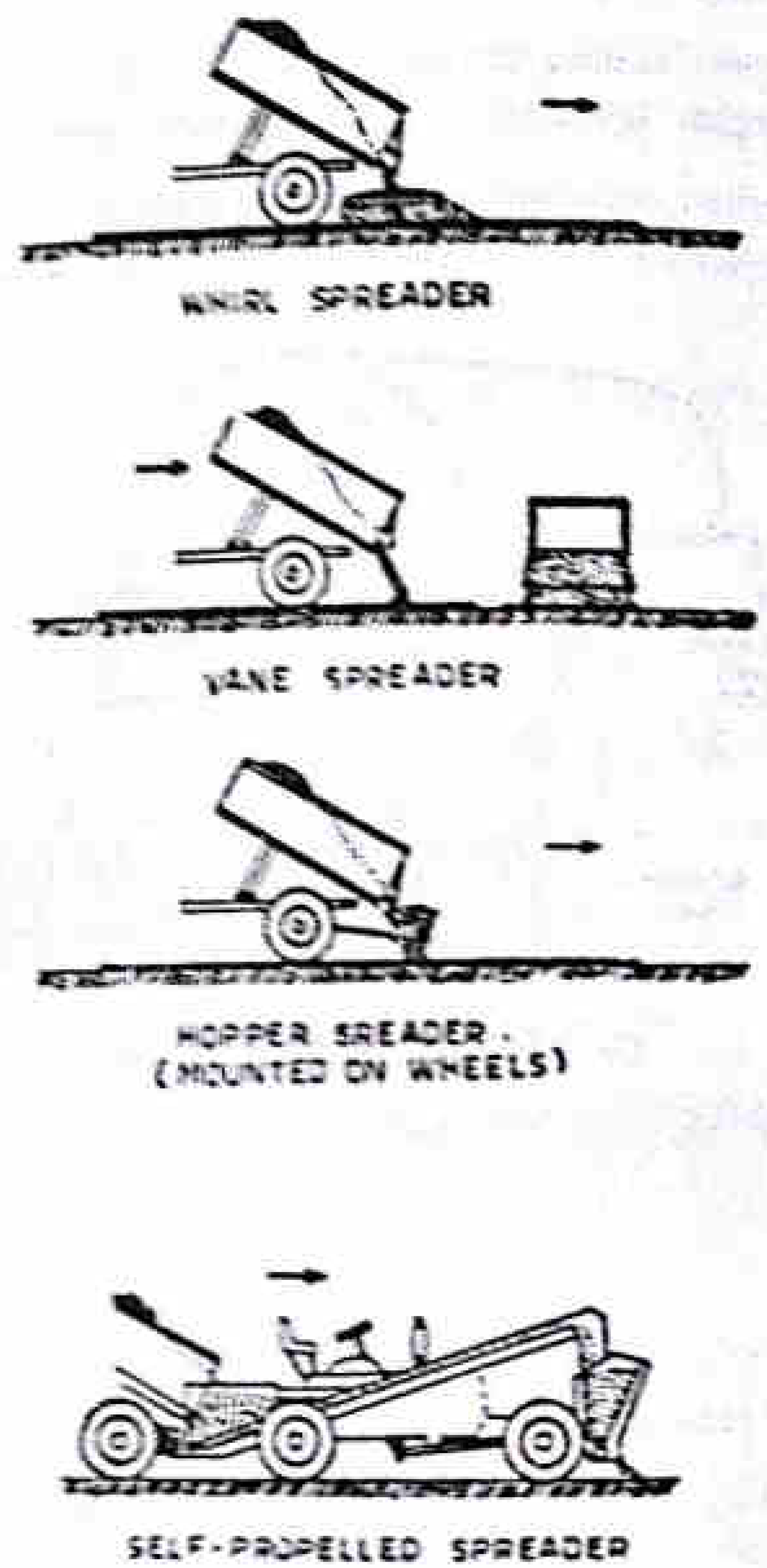


Fig. 8.12 Aggregate Spreaders

(v) *Spreading of key aggregates* : After the application of bitumen, the key aggregates are spread and rolled. The cross profile is again checked.

(vi) *Seal coat* : The seal coat is applied if another surfacing course is not constructed immediately and traffic is to be allowed. Either pre-mixed sand-bitumen or surface dressing type of seal coat may be applied. The pavement section is again rolled. Excessive rolling, however should be avoided.

(vii) *Finishing* : The constructed pavement section is checked for its cross profile with template and longitudinal profile by straight edge. The *unevenness indicator* developed at the Central Road Research Institute, New Delhi, is very useful instrument to locate spots with excessive undulations and thus help in correcting such patches during the construction itself. The maximum permissible undulations on a three metre straight edge is 12 mm and the number of undulations of 10 mm and higher values should not exceed 30 in a road length of 300 m.

(viii) *Opening to traffic* : The finished surface is opened to traffic (if a seal coat or surface dressing has been provided over the penetration macadam) after a minimum period of 24 hours.

Construction of Built-up Spray Grout

Specifications of Materials

Bitumen grades 80/100 to 30/40 or road RT-4 and RT-5 may be used as binder. Aggregates to be used have the same specifications as the penetration macadam, except that when base and binder courses are provided, the base course material may have Los Angeles abrasion value upto 50 percent and aggregates impact value upto 40 percent. The coarse aggregates and key aggregates have similar gradings as penetration macadam of thickness 50 mm.

Construction Steps

(i) *Preparation of existing base* : The depressions and pot holes are filled and the surface is brought to the required grade and camber. The surface is cleaned and a prime coat is applied and cured if the surface is porous.

(ii) *Tack coat* : Heated bituminous binder is applied at a rate of 7.5 to 10 kg on WBM surface and 6 to 8 kg on black top surface per 10 m^2 area.

(iii) *Spreading first layer* : Coarse aggregates are spread and hand packed at a rate of 0.5 m^3 per 10 m^2 area.

(iv) *Rolling* : The aggregate is rolled using 8 to 10 tonnes roller as usual.

(v) *Application of binder* : Bitumen binder is sprayed uniformly at a rate of 12.5 to 15 kg per 10 m^2 over the dry surface of the compacted aggregates.

(vi) *Spreading of second layer of coarse aggregates* : The second layer of coarse aggregates is spread at a rate of 0.5 m^3 per 10 m^2 area, immediately after applying the binder.

(vii) *Rolling* : This layer is again rolled thoroughly starting from the edges.

(viii) *Second application of binder* : Binder is applied at a rate of 12.5 to 15.0 kg per 10 m^2 area.

(ix) *Application of key aggregates* : Immediately after the application of the binder, key aggregates are spread at a rate of 0.13 m^3 per 10 m^2 area and roiled.

(x) *Surface Finish* : The surface unevenness is checked with three-metre straight edge. Longitudinal profile should not have undulations exceeding 12 mm and the number of undulations greater than 10 mm size should not exceed 30 in 300 m length of road. In cross profile the maximum allowable variation in surface is 8.0 mm.

(xi) *Opening to traffic* : Built-up spray grout should not be exposed to traffic before providing surface course.

Construction Procedure for Bituminous Macadam

The Bituminous Macadam (BM) bitumen bound macadam is a premix laid immediately after mixing and then compacted. It is an open graded construction suitable only as a base or binder course. When this layer is exposed as a surface course, at least a seal coat is necessary.

Specifications of Materials :

The grades of bitumen used are 30/40, 60/70 and 80/100 penetration. Road tar RT-4, cutback and emulsion can also be used in cold mix construction technique. The binder content used varies from 3.0 to 4.5 percent by weight of the mix.

Aggregates used are of low porosity fulfilling the following requirements for the base course.

Los Angles abrasion value	50 percent max.
Aggregate impact value	35 percent max.
Flakiness index	15 percent max.
Stripping at 40°C after 24 hours immersion (CRRI test)	25 percent max.
Loss with sodium sulphate, 5 cycles	12 percent max.

For binder course the specified maximum abrasion and impact values are 40 and 35 percent respectively.

The gradings of the aggregates for 75 mm and 50 mm thickness for base and binder course construction as specified by Indian Roads Congress are given in Table 8.4 (a) & (b) respectively.

Table 8.4 Grading of aggregates for bituminous macadam

(a) Grading for 75 mm compacted thickness :

Percent passing sieve size, mm	Base course		Base or binder course
	Grading 1	Grading 2	Grading 3
63.0	100	100	–
50.0	–	90 – 100	–
40.0	35 – 70	35 – 65	100
25.0	–	20 – 40	70 – 100
20.0	0 – 15	–	50 – 80
12.5	–	5 – 20	–
10.0	–	–	25 – 50
4.75	–	–	10 – 30
2.36	0 – 5	0 – 5	5 – 20
0.075	0 – 3	0 – 5	0 – 4
Binder content, percent By weight of mix :	3.0 – 4.5	3.0 – 4.5	3.0 – 6.0

(b) Grading for 50 mm compacted thickness :

Percent passing sieve size, mm	Base course		Base or binder course
	Grading 1	Grading 2	Grading 3
50.0	100	100	–
40.0	–	90 – 100	–
25.0	35 – 70	50 – 80	100
20.0	–	–	70 – 100
12.5	0 – 15	10 – 30	–
10.0	–	–	35 – 60
4.75	–	–	15 – 35
2.36	0 – 5	–	5 – 20
0.075	0 – 3	0 – 5	0 – 4
Binder content, percent by weight of mix :	3.0 – 4.5	3.0 – 4.5	3.0 – 6.0

The quantity of aggregates required for 10 m^2 of bitumen bound macadam are 0.60 to 0.75 m^3 and 0.90 to 1.0 m^3 respectively, for 50 and 75 mm compacted thickness. The bitumen quantity would be determined based on the gradings adopted as specified above.

Plants and Equipment

Various plants and equipment required for the job include :

Sprayer

Mechanical mixer or improvised hand mixer

Spreader – mechanical paver or finisher, grader, or manual methods

Roller

Various other equipment are explained in earlier articles. A typical mechanical finisher is shown in Fig. 8.13. It is a self propelled machine, consisting of a tractor unit and a screeding unit.

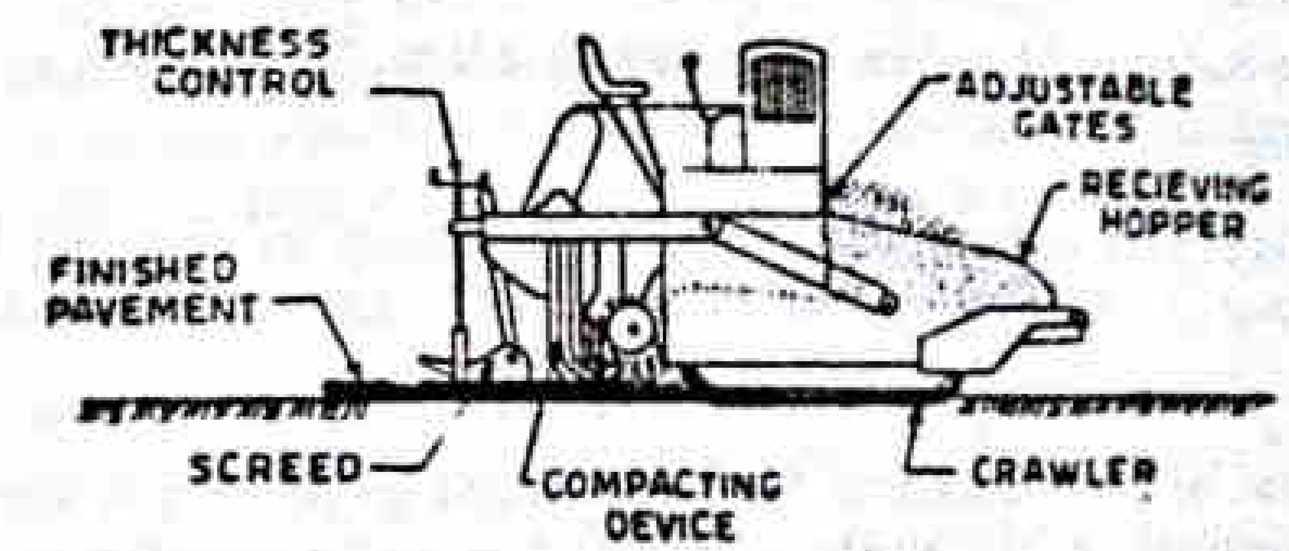


Fig. 8.13 Mechanical Finisher

There are different types of mixer in use. Mechanical mixers include travelling plant or central plant and are employed depending upon the job. Manual hand mixer is also used which is fixed vane type.

Construction Steps

(i) *Preparation of existing layer.* The existing layer is prepared to a proper profile. Pot holes are patched and irregularities are made even. The surface is properly cleaned.

(ii) *Tack coat or prime coat application.* A track coat is applied of thin layer of bitumen binder on the existing layer either using the sprayer or a pouring can. The quantity of application is 4.0 to 7.5 kg per 10 m^2 for black top layer and 7.5 to 10 kg per 10 m^2 for untreated WBM layer.

(iii) *Premix preparation.* The bitumen binder and aggregates as per recommended gradings are separately heated to the specified temperatures and are then placed in the mixer chosen for the job. The mixing temperature for each grading and the bitumen binder is also specified based on the laboratory results. A tolerance of ± 10°C is allowed. The mixing is done till a homogeneous mixture is obtained. The mixture is then carried to the site for its placement through a transporter or a wheel barrow.

(iv) *Placement.* The bituminous paving mixture is then immediately placed on the desired location and is spread with rakes to a pre-determined thickness. The camber profile is checked with a template. It may be stated here that a compacting temperature also influences the strength characteristic of the resulting pavement structure. It is therefore required that the minimum time is spent between the placement of the mix and the rolling operations.

(v) *Rolling and finishing the paving mix.* The rolling is done with 8 to 10 tonnes tandem roller. The rolling is commenced from the edges of the pavement construction towards the centre, and uniform overlapping is provided. The finished surface should not show separate lines of markings due to defective or improper rolling. The roller wheels are kept damp, otherwise the paving mix may partly stick to the wheels and the finishing may not be good. A variation of 6 mm over 3 m length is allowed in the cross profile. The number of undulations exceeding 10 mm should be less than 30 in 300 m length of pavement.

Construction Procedure for Pre-mixed Bituminous Carpet

Specifications of Materials

The bitumen binder of 80/100 grade or road tar of grade RT-3 is used.

The quantity of binder required for 2 cm thick carpet is as under.

(i) Prime coat on a WBM surface 7.3 to 9.8 kg bitumen or 12.20 to 14.64 kg road tar per 10 m^2.

(ii) Tack coat on an existing black top surface 4.9 to 7.3 kg bitumen or 7.3 to 9.8 kg road tar per 10 m^2.

For premixing, depending upon the size and quantity of chipping for both sizes of aggregates, a total of 14.92 kg bitumen binder or 19.73 kg road tar binder is required per 10 m^2 area.

For the carpet of 2 cm thickness, the stone chippings of 12 mm size (i.e. passing 20 mm sieve and retained on 10 mm sieve) and of 10 mm size (passing 12 mm and retained on 6 mm), are required at the rate of 0.183 m^3 and 0.091 m^3 per 10 m^2 area.

The stone chippings should be angular, clean, hard, tough and durable. Sand of clean, hard and durable type is also required.

The aggregates fulfilling the following requirements may be selected :

Los Angles abrasion value	=	35 % max.
(or) Aggregate impact value	=	30 % max.
Flakiness index	=	30 % max.
Stripping value	=	25 % max.
Water absorption	=	2.0 % max.

Plants and Equipment

The various equipment required for this purpose are already discussed in the previous construction types.

Construction steps

(i) Preparation of the existing surface :

Before the carpet is applied to the existing layer, it is necessary to prepare the same by filling the pot holes or depressions with precoated chippings as required. A tack coat is applied over a WBM or an old bitumen surface prior to placing the premix. If the existing surface is of softer aggregates such as laterite, kankar or moorum, first a prime coat is applied and cured.

(ii) Application of tack coat :

The bitumen binder is heated to a specified temperature before its application on the existing pavement surface. The tack coat or prime coat is applied just before spreading the premix.

(iii) Preparation and placing of premix :

The premix is prepared in a mechanical mixer or in an improvised hand drum mixer for the premixing. The capacity of the drum mixer requires to place a quantity of 0.03 m^3 of 12 mm chippings and 0.015 m^3 of 10 mm chippings in one batch. The aggregates and the bitumen are however heated separately upto the required temperature. After the mixing is thorough and homogeneous mix is obtained, the same is taken out and carried at the site for spreading and rolling. The spreading is done with suitable rakes.

The cross profile of the laid material is checked with suitable templates.

(iv) Rolling and finishing :

Immediately after spreading the mix, the rolling operation is commenced. At one operation 15 m of the premix is laid and rolled. A tandem or pneumatic roller of 6 to 9 tonnes is used. To prevent adhesion of the mix to the roller wheels, the same are kept damp with water. The rolling is done until there is no further movement of aggregates in the mix.

(v) Application of seal coat :

In areas of low rainfall, a premixed-sand-seal coat is applied over the carpet. For this purpose a medium coarse sand passing 1.7 mm sieve and retained on 1.18 mm sieve is used at the rate of 0.06 m^3 per 10 m^2. The binder required for this purpose is 6.8 kg per 10 m^2 of the road surface. In areas of high rainfall (over 125 cm per year) a liquid seal is sprayed at 9.8 kg per 10 cm^2 area and covered with a layer of chippings applied over the carpet. The stone chipping of 6 mm size i.e., passing 10 mm sieve and retained on 2.5 mm are used at the rate of 0.09 m^3 per 10 m^2 area. This layer is rolled by a light tandem roller to give a smooth finished surface.

(vi) Surface finish :

The longitudinal undulations under 3.0 m straight edge should not exceed 10 mm and the number of undulations higher than 6.0 mm should not exceed 30 in a length of 300 m road length. The cross profile should not have undulations exceeding 6.0 mm.

(vii) Opening to traffic :

The road may be opened to traffic 24 hours after providing the seal coat or surface dressing.

Construction Procedure for Bituminous Concrete

The bituminous concrete is the highest quality of construction in the group of black top surfaces. Being of high cost specifications, the bituminous mixes are properly designed to satisfy the design requirements of the stability and durability. The mixture contains dense grading of coarse aggregate, fine aggregate and mineral filler coated with bitumen binder. The mix is prepared in a hot-mix plant. The thickness of the bituminous concrete layer depends upon the traffic and quality of base course.

The specifications of materials and the construction steps for bituminous concrete or asphaltic concrete (AC) surface course are given below :

Specification of Materials

(a) Binder : Bitumen of grade 30/40, 60/70 or 80/100 may be chosen depending on the climatic condition of the locality.

(b) Aggregates and filler : The coarse aggregates should fulfill the following requirements.

Aggregate impact value, maximum percent	:	30
or Los Angeles abrasion value, max percent	:	40
Flakiness index, max percent	:	25
Stripping at 40°C after 24 hours, max percent	:	25

Soundness

Loss with sodium sulphate in 5 cycles, max. percent	:	12
Loss with magnesium sulphate in 5 cycles, max. percent	:	18

The gradation of aggregates and filler should conform to those given in Table 8.5.

Table 8.5 Gradation of Aggregates for Bituminous Concrete

Sieve size, mm	Percent passing by weight: Grading 1	Grading 2
20.00		100
12.50	100	80 – 100
10.00	80 – 100	70 – 90
4.75	55 – 75	50 – 70
2.36	35 – 50	35 – 50
0.60	18 – 29	18 – 29
0.30	13 – 23	13 – 23
0.15	8 – 16	8 – 16
0.075	4 – 10	4 – 10

(c) Bituminous concrete mix :

Marshall Stability Test number of blows to be applied on either side of specimen : 50

Marshall stability value, minimum kg	:	340
Marshall Flow value, 0.25 mm units	:	8 to 16
Voids in mix, percent	:	3 to 5
Voids filled with bitumen, percent	:	75 to 85

Plants and Equipment

For construction work of the bituminous concrete, a hot mix plant is used. In order to achieve high quality in construction mechanised construction equipment should be used. A mechanical finisher may be used if available. The finisher is designed to produce a uniform level riding surface and to correct automatically small irregularities in the pavement surface. It consists of a receiving hopper, arrangements for spreading the mix to a desired uniform thickness and a tamper. This equipment is considered to be important in the construction of bituminous concrete pavements.

Construction Stage

(i) Preparation of the existing base course layer :

The existing surface is prepared by removing the pot holes or ruts if any. The irregularities are filled in with premix chippings at least a week before laying surface course. If the existing pavement is extremely wavy, a bituminous levelling course of adequate thickness is provided to lay a bituminous concrete surface course on a binder course instead of directly laying it on a WBM.

(ii) Application of Tack Coat :

It is desirable to lay AC layer over a bituminous base or binder course. A tack coat of bitumen is applied at 6.0 to 7.5 kg per 10 m^2 area, this quantity may be increased to 7.5 to 10 kg for non-bituminous base.

(iii) Preparation and placing of premix :

The premix is prepared in a hot mix plant of a required capacity with the desired quality control. The bitumen may be heated upto 150 – 177°C and the aggregate temperature should not differ by over 14°C from the binder temperature. The hot mixed material is collected from the mixer by the transporters, carried to the location and is spread by a mechanical paver at a temperature of 121 to 163 °C. The camber and the thickness of the layer are accurately verified. The control of the temperatures during the mixing and the compaction are of great significance in the strength of the resulting pavement structure.

(iv) Rolling :

A mix after it is placed on the base course, is thoroughly compacted by rolling at a speed not more than 5 km per hour. The initial or break down rolling is done by 8 to 12 tonnes roller and the intermediate rolling is done with a fixed wheel pneumatic roller of 15 to 30 tonnes having a tyre pressure of 7 kg per cm^2. The wheels of the roller are kept damp with water. The number of passes required depends on the thickness of the layer. In warm weather rolling on the next day helps to increase the density if the initial rolling was not adequate. The final rolling or finishing is done by 8 to 12 tonne tandem roller.

(v) Quality control of bituminous concrete construction

The routine checks are carried out at site to ensure the quality of the resulting pavement mixture and the pavement surface. Periodical checks are made for (a) aggregate grading (b) grade of bitumen (c) temperatures of aggregate (d) temperatures of paving mix during mixing and compaction. At least one sample for every 100 tonnes of the mix discharged by the hot mix plant is collected and tested for above requirements. Marshall tests are also conducted. For every 100 m^2 of compacted surface, one test of the field density is conducted to check whether it is at least 95% of the density obtained in the laboratory. The variation in thickness allowed is 6 mm per 4.5 m length of construction.

(vi) Finished surface :

The AC surface should be checked by a 3.0 metre straight edge. The longitudinal undulations should not exceed 8.0 mm and the number of undulations higher than 6.0 mm should not exceed 10 in a length of 300 metre. The cross profile should not have undulations exceeding 4.0 mm.

8.7 CONSTRUCTION OF CEMENT CONCRETE PAVEMENTS

8.7.1 Introduction

The cement concrete pavement maintains a very high recognition among the engineer and the road users alike. Due to the excellent riding surface and pleasing appearance, the cement concrete roads are very much preferred. Further the engineers have inherent confidence in the cement concrete materials for its use in any construction project. It is also true that the life of a cement concrete road is much more than any other type of construction. Further, the cement concrete material exhibits its characteristics which can be predicted by elastic theory and as such a structure made using this material can well be designed on a rational basis. This indirectly saves cost as the resultant structure gives an excellent performance.

In considering the drawbacks of the cement concrete pavement type, it may be stated that it requires a very high initial investment and the method is not adopted for the stage construction. Transverse and longitudinal joints are unavoidable in this construction. The joints as such provide additional planes of weaknesses. The maintenance and repair, if any, required in cement concrete pavements are mostly associated with joints. The number of joints provided are also kept to a minimum as the construction of joints involve substantial extra work. A minimum period of 28 days curing is required before the cement concrete pavement could be opened to the traffic. The cement concrete pavements are constructed with or without the sub-base course. This decision is made depending upon the soil type, design load and economic consideration. The various purpose of the sub-base course beneath the cement concrete pavements are :

(i) to provide a strong supporting layer.

(ii) to provide a capillary cut-off preventing the damages due to *mud pumping*.

(iii) to reduce thickness requirements of cement concrete slab and lower the cost of construction. WBM is the most popular type of underlying layer generally adopted in India. Soil stabilized layers can also be used with advantage.

(iv) to increase the service life of the CC pavement

The construction of cement concrete pavement is dealt under the following groups :

1. Construction of pavement slabs
2. Construction of joints

8.7.2 Construction of Cement Concrete Pavement Slab

Various specifications for construction of cement concrete pavements are listed below.

Cement grouted layer

Rolled concrete layer

Cement concrete slab

In cement grouted layer, open graded aggregate mix with minimum size of aggregates as 18 to 25 mm is laid on the prepared subgrade and the aggregates are dry rolled. The loose thickness is compacted to provide 80 percent of rolled thickness. The grout made of coarse sand, cement and water is prepared. The proportion of cement to sand is taken as 1 : 1½ to 1 : 2½. To provide proper fluidity to the grout, wetting agent is also added to the mix. The grout is applied on the surface and is allowed to seep through the

aggregate matrix. The technique can be compared with bituminous grouted or penetration type construction.

In rolled concrete layer, lean mix concrete is used. Lean mix of aggregate, sand, cement and water is prepared and laid on the prepared, subgrade or sub-base course. The rolling is done similar to WBM construction. The loose thickness of the concrete is 20 percent more than the compacted or finished thickness. Tandem rollers are recommended. The rolling operation is completed before the final setting time of cement. Curing is done as per conventional method.

Cement grouted and rolled concrete are suitable for base course only. The construction details of cement concrete slabs are given in the following paragraphs, as they serve as both base and surface courses of highway pavements.

There are two modes of construction of cement concrete slab

(i) Alternate bay method

(ii) Continuous bay method

Alternate bay method of construction means constructing a bay or one slab in alternate succession leaving the next or intermediate bay to follow up after a gap of one week or so.

See Fig. 8.14. In alternate bay construction the slabs constructed are in sequence of X, Y, Z etc. leaving gaps of bay X′, Y′, Z′, etc. This technique provides additional working convenience for laying of slabs. The construction of joints is easier.

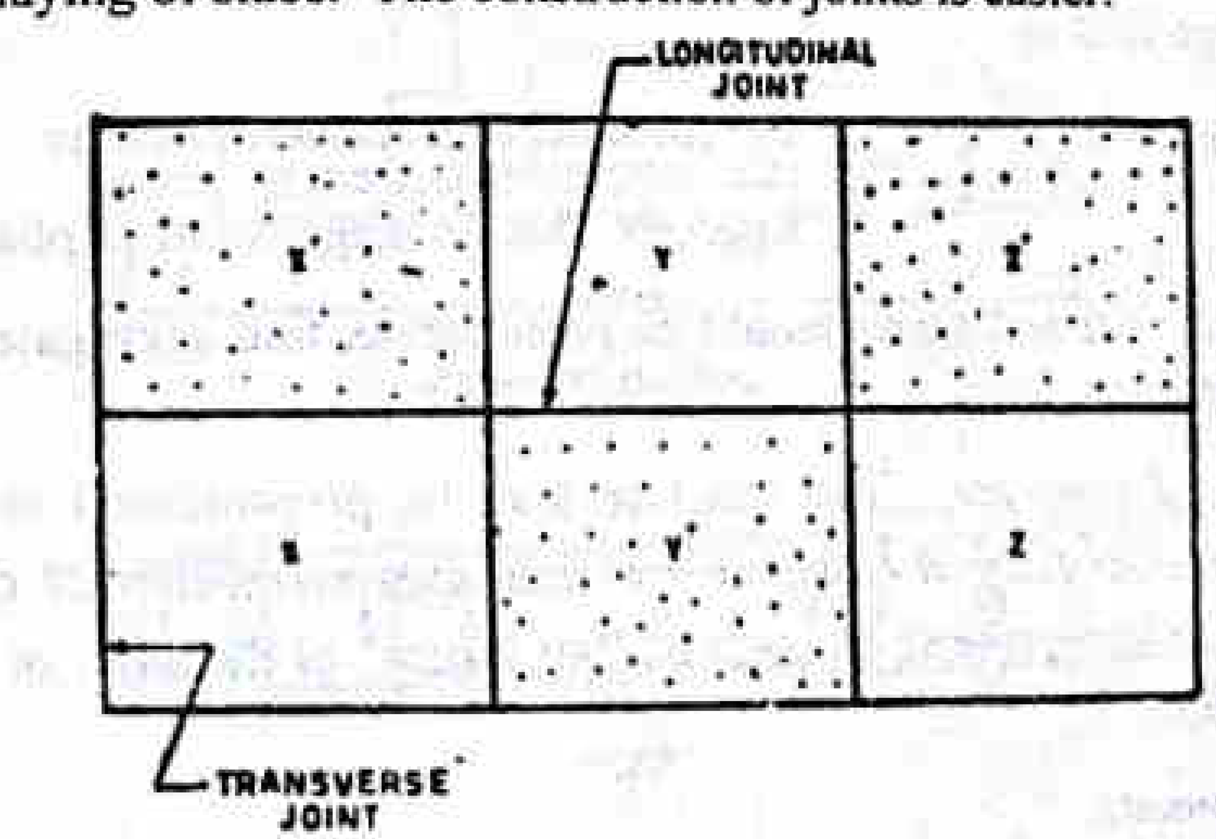


Fig. 8.14 Construction Method of Cement Concrete Road

But it has many drawbacks as follows :

(i) Large number of transverse joints are to be provided. This increases the construction cost and reduces the smooth riding quality of the surface.

(ii) During rains, the surface water collects on the subgrade between the finished bays.

(iii) The construction is spread over the full width of road and the traffic will have to be completely diverted.

In continuous (bay) method all the slabs or bays are laid in sequence i.e. X′, Y, Z′ etc. or X, Y′, Z etc. Construction joints are however provided at the end of the day's job.

In general the later method is preferred mainly because of the advantage that construction of half the pavement width can be taken at a time while essential traffic could be diverted on the other half of the road.

Specifications of material for cement concrete pavement slabs. The materials required for plain concrete slabs are cement coarse aggregates, fine aggregates and water. In case reinforcement is provided, steel wire fabric or bar mats may be used of the required size and spacing. Other materials are for the construction of joints, such as load transfer devices, joints filler and sealer.

Cement : Ordinary portland cement is generally used. In case of urgency rapid hardening cement may also be used to reduce curing time.

Coarse aggregates : The maximum size of coarse aggregates should not exceed one fourth the slab thickness. The gradation of coarse aggregate may range from 50 to 4.75 or 40 to 4.75 mm. the aggregate is collected in two size ranges, one below and the other above 20 mm size. When the grading is from 20 to 50 mm, the materials are collected in two groups, below and above 25 mm size. The aggregate should be free from harmful materials such as iron, pyrites, coal, mica, clay, alkali, organic impurities etc. The coal, lignite, clay or fines passing 75 micron sieve in the coarse aggregates should not be more than one percent by weight. The desirable limits of important properties are :

Aggregate crushing value	:	30 % max.
Aggregate impact value	:	30 % max.
Los Angeles abrasion value	:	30 % max. as per the ISI and
	:	35% max. as per the IRC
Soundness, average loss in weight after 10 cycles	:	12 percent max. in sodium sulphate
	:	18 percent max. in magnesium sulphate

Fine aggregate. Natural sands should be preferred as fine aggregate though crushed stones may also be used.

Proportioning of concrete. The concrete may be proportioned so as to obtain a minimum modulus of rupture of 40 kg/cm^2 on field specimens after 28 days curing or to develop a minimum compressive strength of 280 kg/cm^2 at 28 days, or higher value as desired in the design.

Plants and Equipment

The equipment necessary for the construction of cement concrete slabs are for batching, mixing, placing, finishing and carrying the concrete pavement. Equipment commonly used are given below.

Concrete Mixer

If batching by volume is required then the separate measuring boxes are provided for the different aggregates. Each box is provided with a straight edge for striking off excess material after filling.

Batching Device

Concrete mixer of adequate capacity of the batch type is provided. It has a rated capacity of not less than 0.2 m^3 of mixed concrete. The mixture is equipped with a water measuring device capable of accurate measurement of water required per batch. Some mixers are also equipped with timing devices which automatically lock the discharge lever during the full time of mixing and releases it at the end of mixing period.

Wheel Borrow

Wheel borrows with two wheels are used to transport concrete for short distances from the mixer.

Vibrating Screed

Vibrating screed comprises of a wooden or mild steel screed with suitable handles driven by vibrating units mounted thereon, propelled either electrically or by compressed air or by a petrol engine, and travelling on side forms.

Internal Vibrators

It comprises of vibrating head with suitable motive power either of compressed air, electricity or of a petrol driven engine right enough to ensure proper control and manipulation in the mass of concrete. It is used to ensure compaction of the cement concrete along with the forms and also to avoid any tendency of honey-combing at the edges of the slab.

Tools and appliances for surface, finishing operations in common use are float, straight edge, belt and fibre brush.

Float

The longitudinal float is of 75 cm length and 7.5 cm width and is made of hard wood and is fixed with handle. (See Fig. 8.15). This is used for smoothing the concrete.

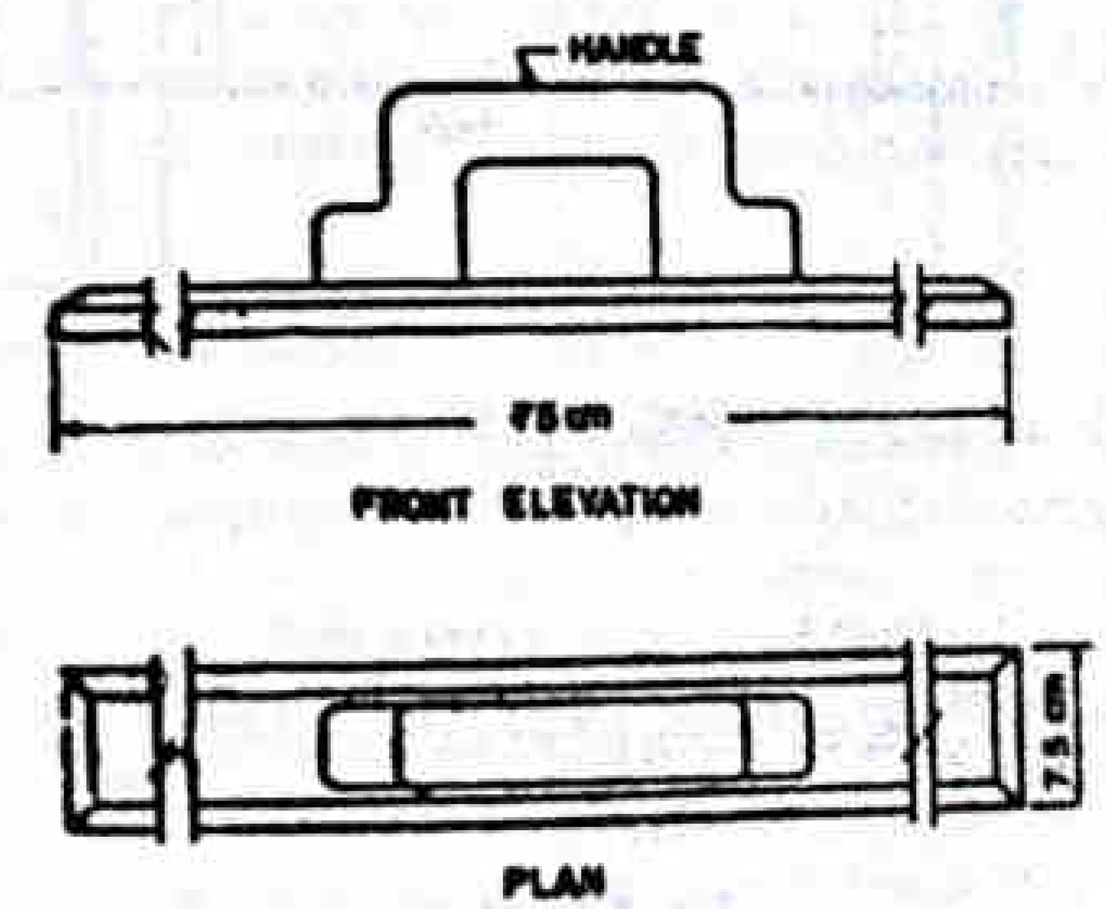


Fig. 8.15 Float

Straight Edge

It is used to check the finished pavement surface in longitudinal direction. It is made of hard wood with M.S. plate at bottom, 3 meter in length, 10 cm in width with two handles as shown in Fig. 8.16.

Belt

Canvas belts are used for finishing the pavements surface before the concrete hardens. The canvas is of 25 cm width and atleast 75 cm longer than the width of pavement slab. It has two wooden handles at the end. See Fig. 8.17.

Fibre Brush

Fibre brush broom is used to make broom marks across the pavement surface and to make it skid resistance. Hard fibres are used projecting out of the wooden brush of length 45 cm and width 7.5 cm, with a handle about 2 meter long

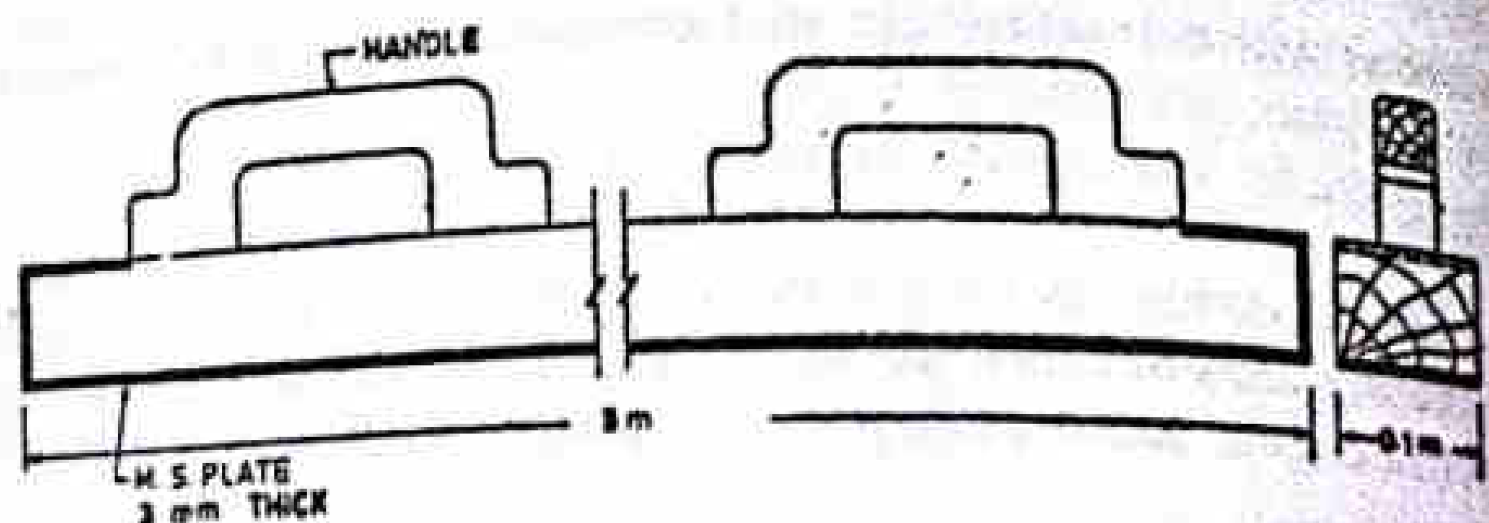


Fig. 8.16 Straight Edge

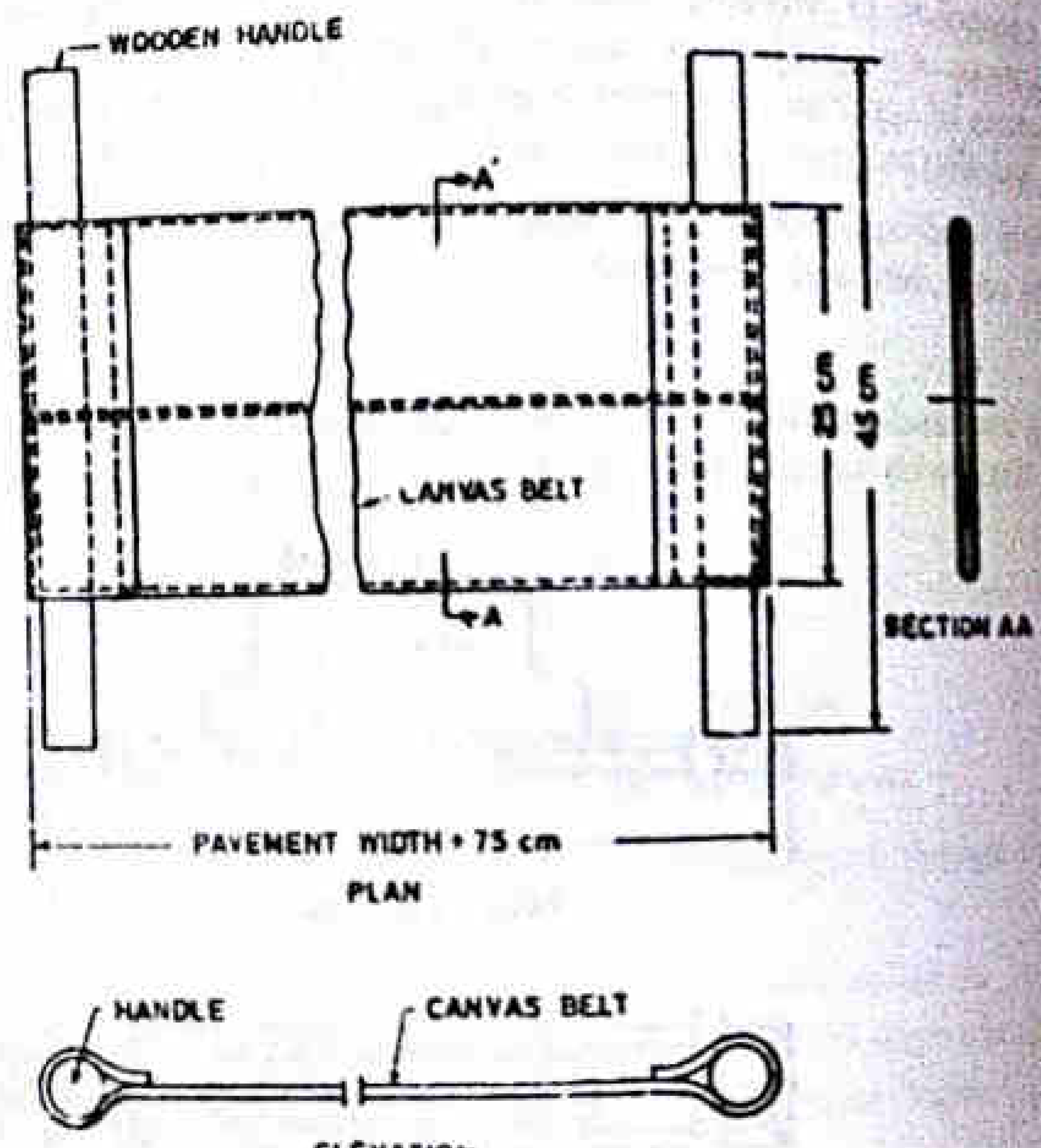


Fig. 8.17 Belt

Edging Tool

The edging tool is used for rounding the transverse edges at expansion joints and the longitudinal edges. The vertical limb of this tool extends to the required depth. The rounded edge of the M.S. plate has radius of 6 mm.

Other Small Tools

Other small tools and equipment such as spades, shovels and pans water pots etc necessary for the work are also provided.

Construction steps for cement concrete pavement slab

(i) Preparation of Subgrade and Sub-base

The subgrade or sub-base for laying of the concrete slabs should comply with the following requirement; that no soft spots are present in the subgrade or sub-base; that the

uniformly compacted subgrade or sub-base extends atleast 30 cm on either side of the width to be concreted; that the subgrade is properly drained; that the minimum modulus of subgrade reaction obtained with a plate bearing test is 5.54 kg/cm^2

The subgrade is prepared and checked at least two days in advance of concreting. The subgrade or sub-base is kept in moist condition at the time when the cement concrete is placed. If necessary, it should be saturated with water for 6 to 20 hours in advance of placing concrete. Water proof paper may also be placed whenever the cement concrete is laid directly over the soil subgrade. In such a case, the moistening of the subgrade prior to placing of the concrete is not required.

(ii) Placing of Forms

The steel or wooden forms are used for the purpose.

The steel forms are of M.S. channel sections and their depth is equal to the thickness of the pavements. The sections have a length of at least 3 m except on curves of less than 45.0 m radius, where shorter sections are used. When set to grade, the maximum deviation of the top surface of any section from a straight line is not exceeded by 3 mm.

Wooden forms are dressed on side; these have minimum base width of 10 cm for slab thickness of 20 cm and minimum base width of 15 cm for slabs over 20 cm thick. The forms are jointed neatly and are set with exactness to the required grade and alignment. Sufficient rigidity is obtained to support the forms in such a position during the entire operation of compacting and finishing that they do not deviate more than 3 mm from a straight edge 3 m in length.

(iii) Batching of Material and Mixing

After determining the proportion of ingredients for the field mix, the fine aggregates and coarse aggregates are proportioned by weight in a weight-batching plant and placed into the hopper along with the necessary quantity of cement. Cement is measured by the bag. All batching of material is done on the basis of one or more whole bags of cement, the weight of one bag is taken as 50 kg or the unit weight of cement is taken as 1440 kg/m^3.

The mixing of concrete is done in batch mixer which will ensure a uniform distribution of the materials throughout the mass, so that the mix is uniform in colour and is homogenous.

The batch of cement, fine aggregate and coarse aggregate is led together into the mixer. The water for mixing is introduced into the drum within the first 15 seconds of mixing. The mixing of each batch is commenced within one and half minute after all the materials are placed in the mixer.

(iv) Transporting and Placing of Concrete

The cement concrete is mixed in quantities required for immediate use and is deposited on the soil subgrade or sub-base to the required depth and width of the pavement section within the form work in continuous operation. Care is taken to see that no segregation of materials results while the concrete is being transported from the mixer to its placement. The spreading is done uniformly. A certain amount of re-distribution is done with shovels. Needle vibrator is employed in lieu of rodding and splicing of the concrete.

(v) Compaction and Finishing

The surface of pavement is compacted either by means of a power-driven finishing machine or by a vibrating hand screed. For areas where the width of the slab is very small as at the corner of road junctions, etc., hand consolidation and finishing may be adopted :

(a) Concrete as soon as placed, is struck off uniformly and screeded to the crown and cross-section of the pavement to conform the grade.

(b) The tamper is placed on the side forms and is drawn ahead in combination with a series of lifts and drops to compact the concrete.

Floating and Straight Edging

The concrete is further compacted by means of the longitudinal float. The longitudinal float is held in a position parallel to carriageway centre line and passed gradually from one side of the pavement to the other. After the longitudinal floating is done and the excess water gets disappeared, the slab surface is tested for its grade and level with the straight edge.

Belting, Brooming and Edging

Just before the concrete becomes hard, the surface is belted with a two-ply canvas belt. The short strokes are applied transversely to the carriageway.

After belting, the pavement is given a broom finish with fibre broom brush. The broom is pulled gently over the surface of the pavement transversely from edge to edge. Brooming is done perpendicular to the centre line of the pavement.

Before the concrete develops initial set, the edges of the slab are carefully finished with an edging tool.

(vi) Curing of Cement Concrete

The entire pavement surface of the newly laid cement concrete is cured in accordance with the following methods :

Initial curing. The surface of the pavement is entirely covered with burlap, cotton or jute mats. Prior or being placed, the mats are thoroughly saturated with water and are placed with the wet side down to remain in intimate contact with the surface.

Final curing. The final curing is done with any one of the following methods :

Curing with wet soil exposed edges of the slab are banked with a soil berm. A blanket of sandy soil free from stones is placed. The soil is thoroughly kept saturated with water for 14 days.

Impervious Membrane Method. Use of an impervious membrane which does not impart a slippery surface to the pavement is used. Liquid is applied under pressure with a spray nozzle to cover the entire surface with a uniform film. It hardens with 30 minutes after its application. The liquid is applied immediately after the surface finishing.

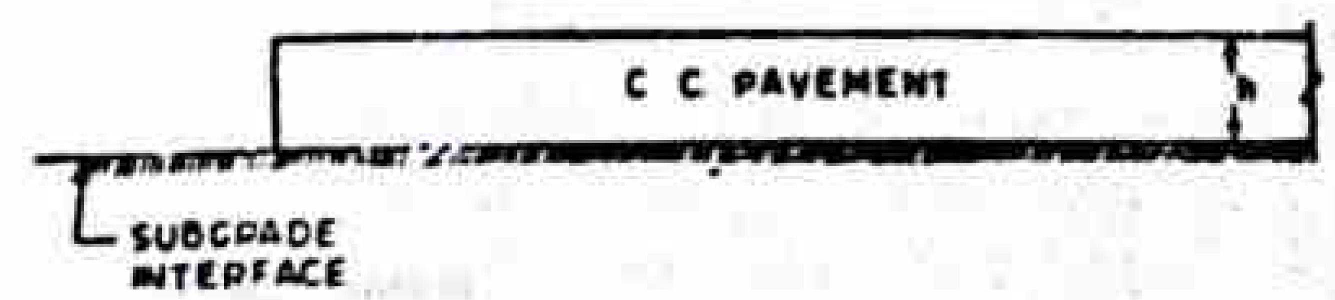
When the concrete attains the required strength or after 28 days of curing the concrete road is opened to the traffic.

8.8 CONSTRUCTION OF JOINTS IN CEMENT CONCRETE PAVEMENTS

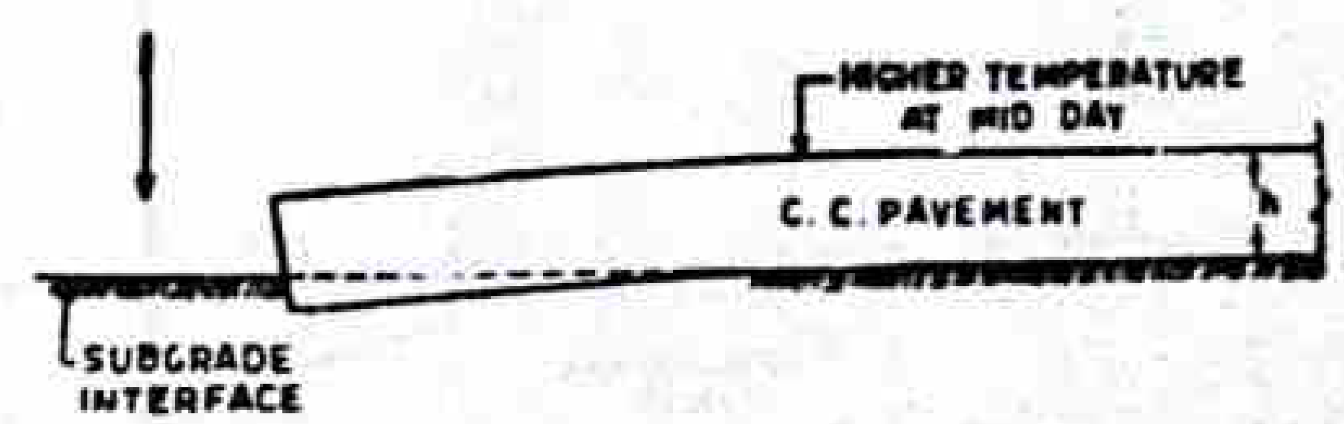
8.8.1 Introduction

Joints are provided in cement concrete roads for expansion, contraction and warping of the slabs due to the variation in the temperature of slabs. Changes in atmospheric temperatures in turn reduce the changes in the temperature of slabs. Such changes of temperature cause expansion of the slab horizontally if there is an increase in the slab temperature above the temperature during which the slab was laid. Similarly there is contraction of slab also when the temperature falls below this temperature. Thus the rise and falls of atmospheric temperatures which is a cyclic phenomenon make the pavement slabs also to expand and contract.

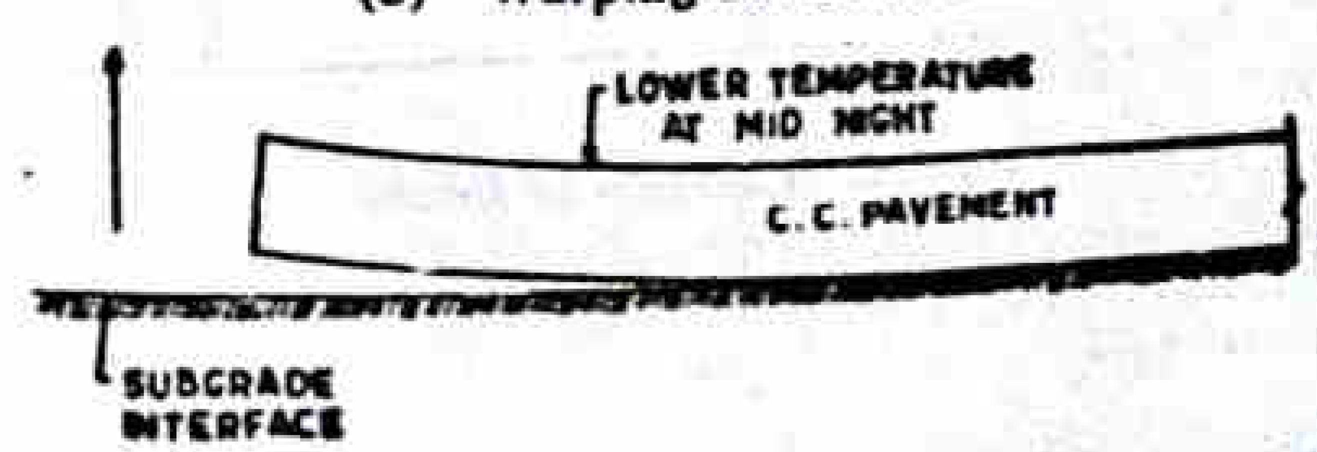
The slab movements also take place in vertical direction which is due to the temperature differential between top and bottom of pavement slab. During the mid-day the top of the pavement slab has higher temperature than the bottom of the slab. This causes the top fibres of the slab to expand more than the bottom fibres, and the slab curls at the edges as shown in Fig. 8.18 b. This phenomenon is known as *warping down* of the slab.



(a) No warping condition



(b) Warping down condition



(c) Warping up condition

Fig. 8.18 Warping of Cement Concrete Pavement Slab

By about the mid night the temperature of the bottom of the slab is higher than the temperature of the slab top. The slab *warps up* during this time. See Fig. 8.18c.

In reality, the weight of the pavement slab prevents the slab to take a warped shape thereby developing stress in the slab which are known as *warping stresses*. The magnitude of warping stresses are maximum at the interior region and are minimum at the corner region.

When the slab is warped down in the day time, the warping stresses are tensile in nature at the bottom of the slab. In warped up condition, the tensile stresses are developed at the top of the pavement slab. To minimise the temperature stresses in the pavement slab, expansion, contraction and warping joints may be provided transversely across the full width of pavement.

In addition to this, the construction joints are also provided. The compulsory break provided in continuity of the slabs is due to the close of day's job and the commencement of the same the next day with a construction joint. Normally the construction joint is planned to coincide with an expansion joint. It is customary to provide concreting of one lane at a time which may be of width 3.5 m for highway pavements. Thus two lanes are also jointed together by a joint known as longitudinal joints. Joints are also classified depending upon their direction of placement :

(i) *Transverse joints*. These are further classified as :

(a) Expansion joint

(b) Contraction joint

(c) Warping joint

(d) Construction joint

(ii) *Longitudinal joints*. The location of these joints are shown in Fig. 8.19.

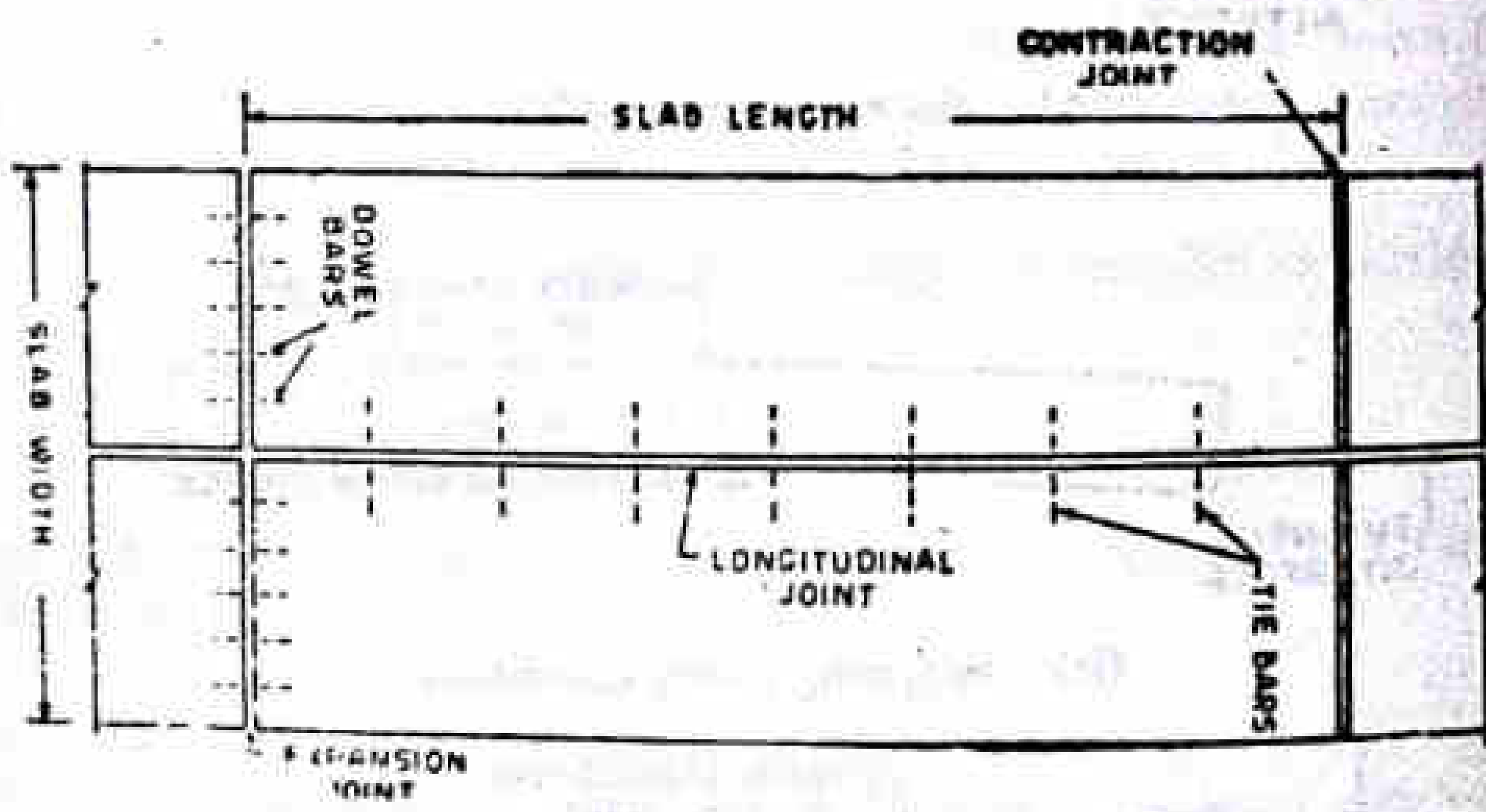


Fig. 8.19 Location of Joints

Following are the requirement of a *good joint* :

(a) Joint must move freely.

(b) Joint must not allow infiltration of rain water and ingress of stone grits.

(c) Joint must not protude out the general level of the slab.

8.8.2 Transverse Joints

Expansion joints

These joints are provided to allow for expansion of the slabs due to rise in slab temperature above the construction temperature of the cement concrete. Expansion joints also permit the contraction of slabs. Expansion joints in India are provided at interval of

50 to 60 metre for smooth interface laid in winter and 90 to 120 metre for smooth interface laid in summer. However for rough interface the spacing between expansion joints may be 140 m. A typical expansion joint is shown in Fig. 8.20. The approximate gap width for this type of joints is from 20 to 25 mm.

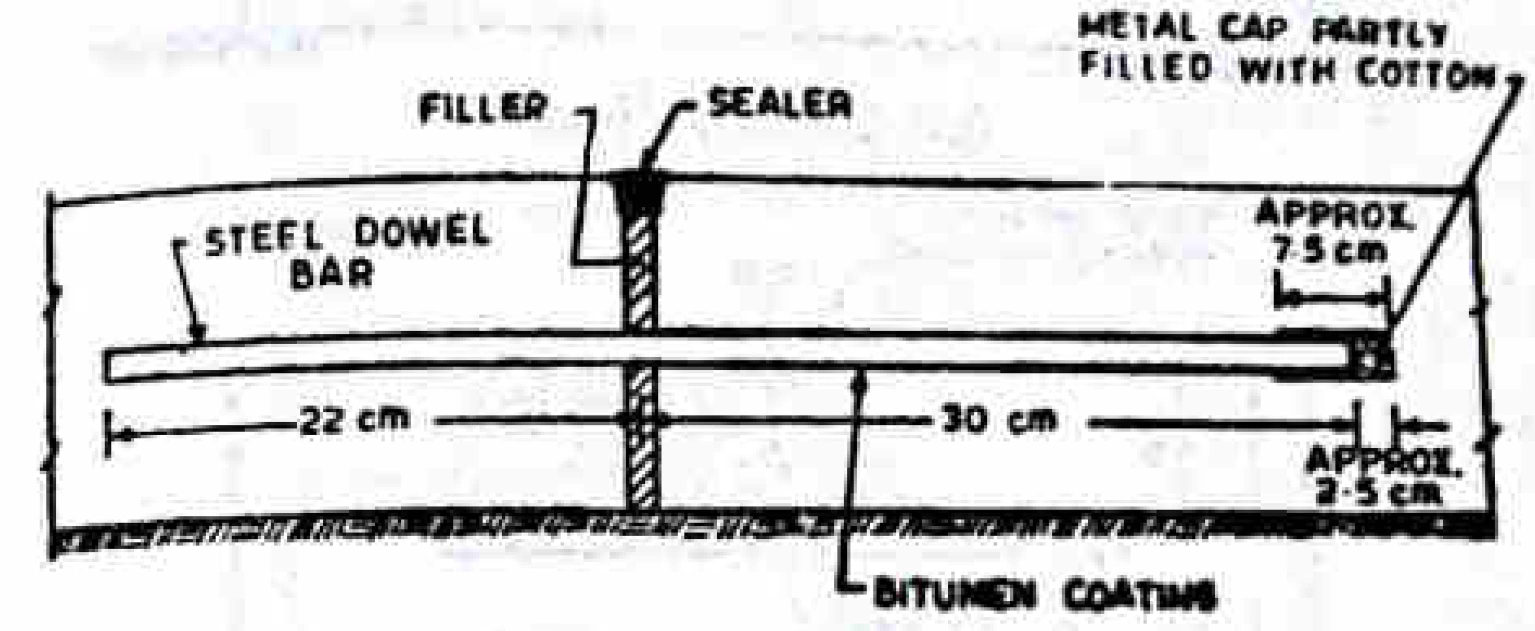


Fig. 8.20 Expansion Joint with Dowel Bar

It may be stated here that the break in slab continuity forming a joint adds a weaker plane in the cement concrete pavement. The stresses include due to the wheel loads at such joints are of very high order at the edge and corner regions. In order to strengthen these locations following measures are adopted :

The load transference across the transverse joint is carried out through a system of reinforcement provided at suitable intervals projecting in the concrete in longitudinal direction upto 60 cm length. Such a device is named as *dowel bar*. See Fig. 8.20. In the expansion joint, thus load transference is affected through a system of dowel bars. Dowel bars are embedded and kept fixed in concrete at one end and the other end is kept free to expand or contract by providing a thin coating of bitumen over it. Metal cap is provided at this end to offer a space of about 2.5 cm for movements during expansion. In the design, 40 percent of wheel load is expected to be taken up by the group of dowel bars and transferred to the adjoining slab. Spacing between the dowel bars is generally adopted as 30 cm. The size of the dowel varies with pavement thickness and it ranges between 20 to 30 mm. The total length of dowel bar varies between 40 cm to 73 cm depending upon the dowel diameter.

Contraction Joints

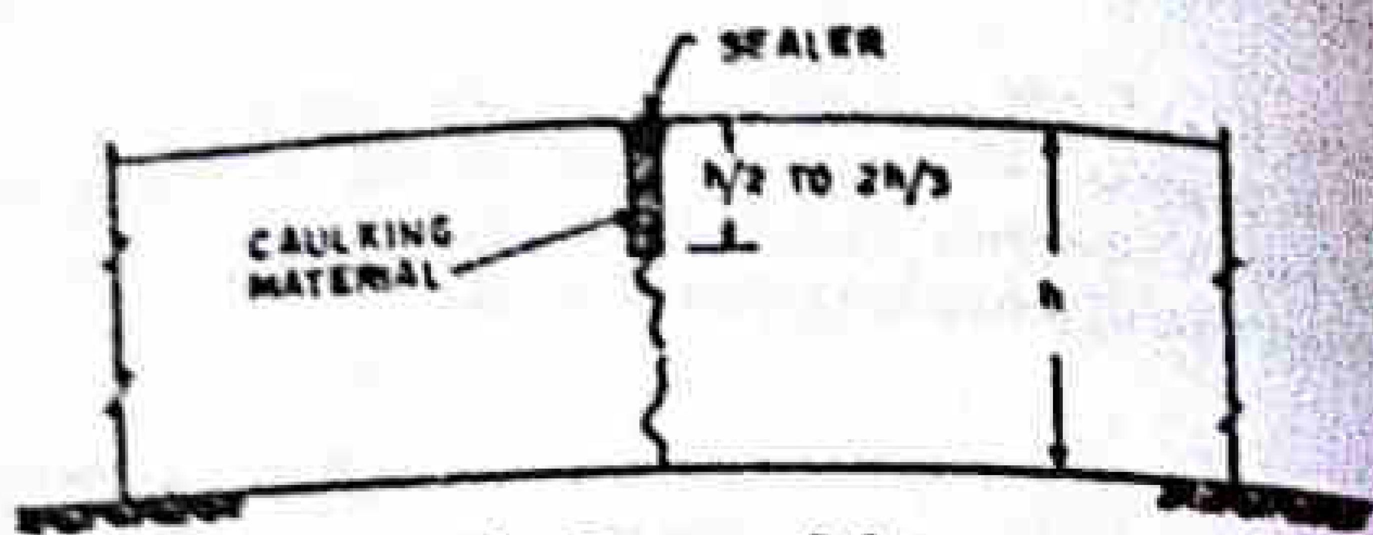
Contraction joints are provided to permit the contraction of the slab. These joints are spaced closer than expansion joints. Load transference at the joints is provided through the physical interlocking by the aggregates projecting out at the joint faces. As per IRC specifications, the maximum spacing of contraction joints in unreinforced CC slabs is 4.5 m and in reinforced slab of thickness 20 cm is 14 m.

Since it is recommended to provide contraction joints at close spacing, there seems to be no need of providing any load transference, as mainly this will be done by the aggregate interlocking. For added safety, some agencies recommended the use of dowel bar which are fully bounded in the concrete.

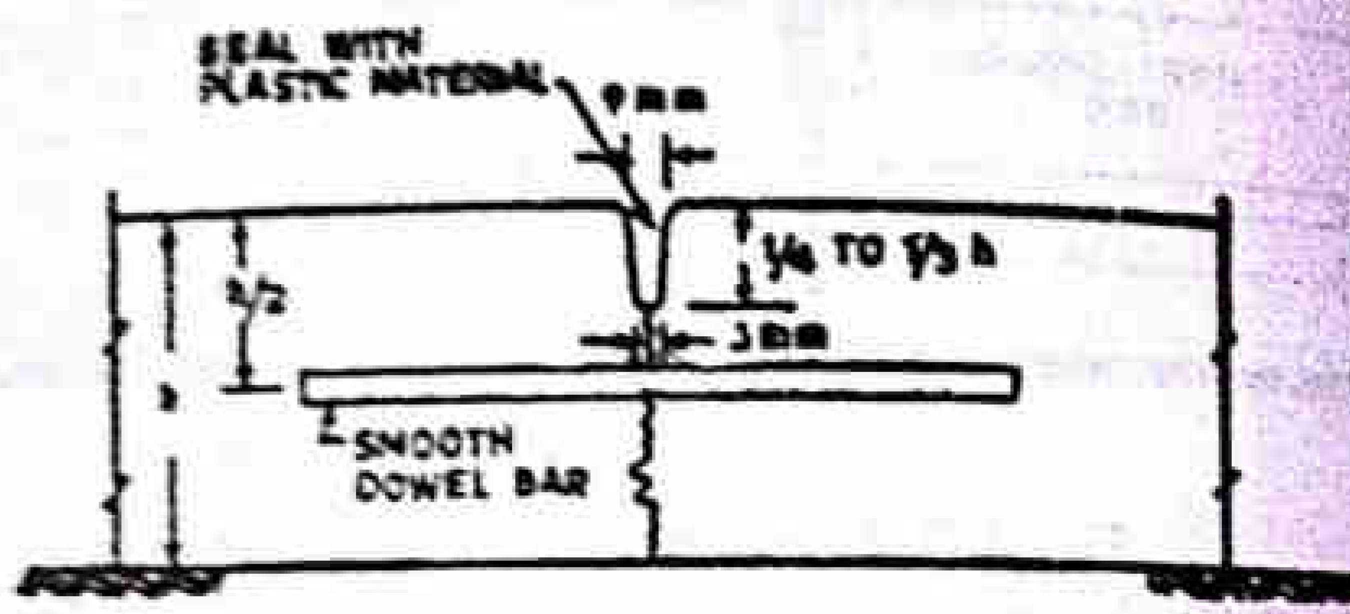
Typical contraction joints are shown in Fig. 8.21.

Warping Joints

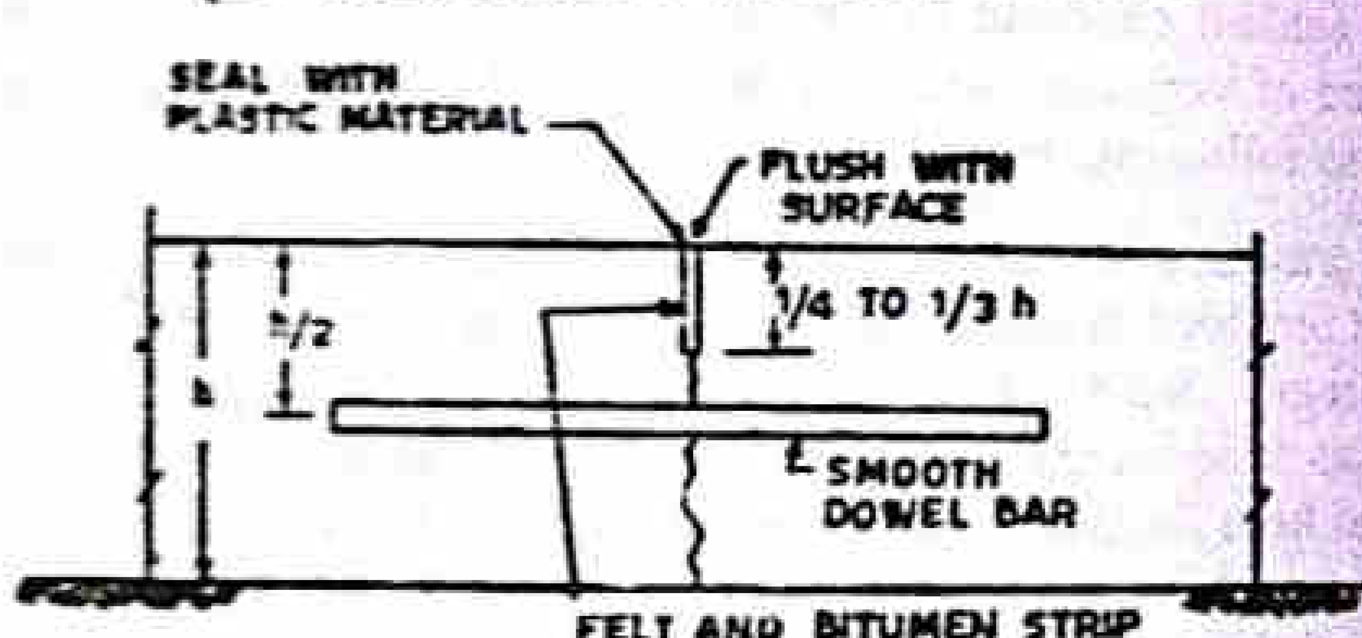
The warping joints are provided to relieve stresses included due to warping. These are known as *hinged joints*. Longitudinal joints with tie bars fall in this class of joint. These joints are rarely needed if the suitably designed expansion and contraction joints are provided to prevent cracking.



(a) Dummy Joint



(b) Contraction Joint with Dowel Bar



(c) Contraction Joint with Dowel Bar

Fig. 8.21 Contraction Joints

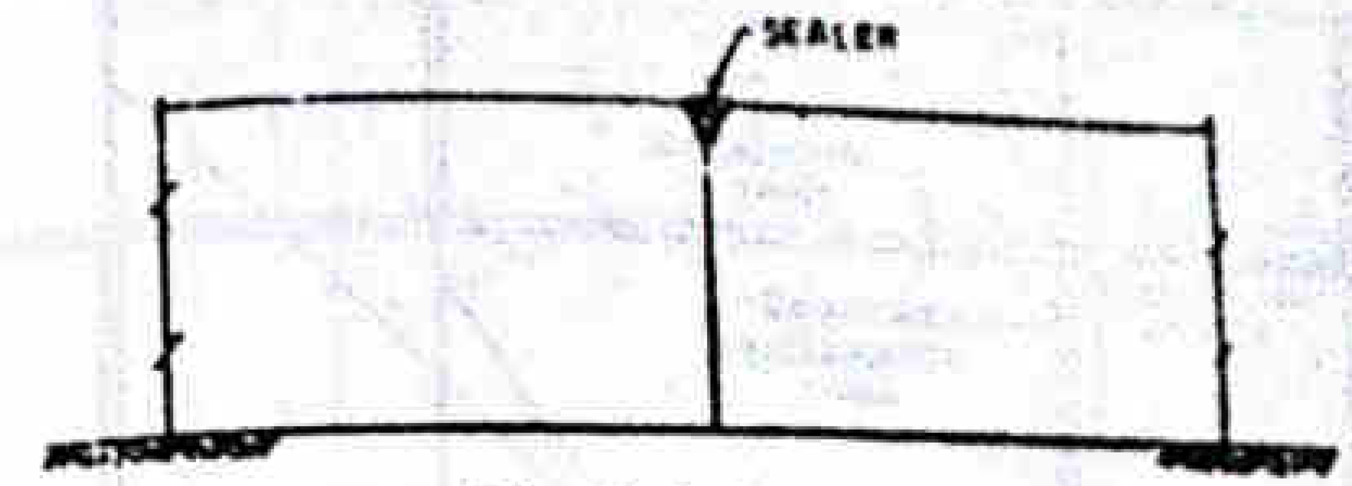
8.8.3 Longitudinal Joints

Longitudinal joints are provided in cement concrete roads which have width over 4.5 m. On soil subgrade of clay, such joints are provided to allow differential shrinkage and swelling due to rapid changes in subgrade moisture under the edges than under the centre of road. The longitudinal joints are provided to prevent longitudinal cracking in the cement concrete pavements. This type of joint acts as a hinge and helps to maintain the two slabs together, at the same level.

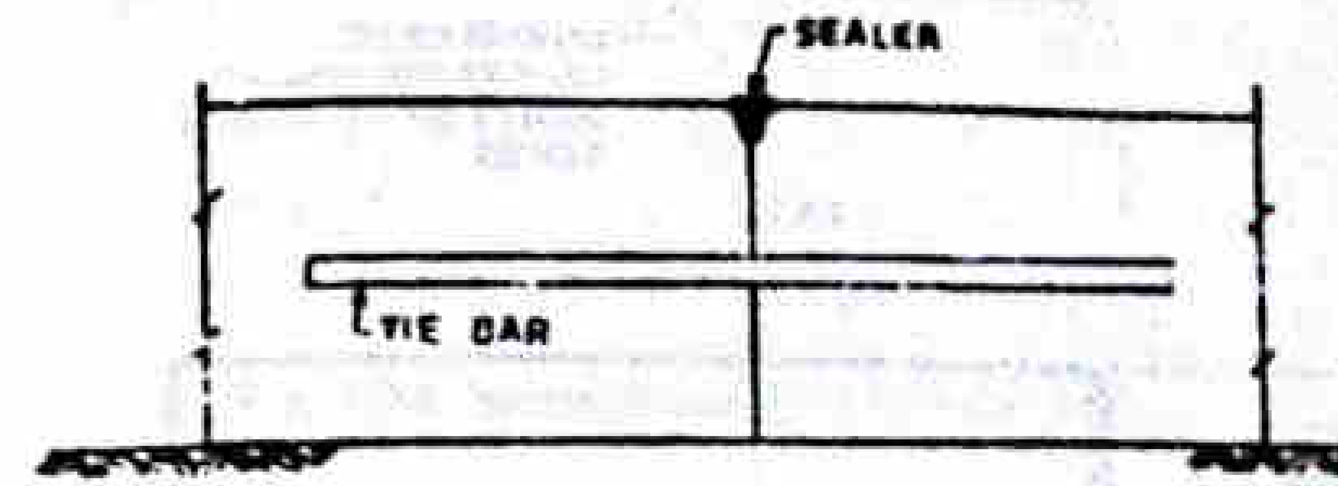
In the longitudinal joints *tie bars* are provided to hold the adjacent slabs together. The various types of longitudinal joints in use are shown in Fig. 8.22. IRC recommends to use plain butt with tie bar type of joints.

A butt joint is the simplest longitudinal joint and is formed by painting the joint face with bitumen. The tie bars are also placed in the type of butt joint with tie bar. Recommended size and spacing of tie bars are as follows :

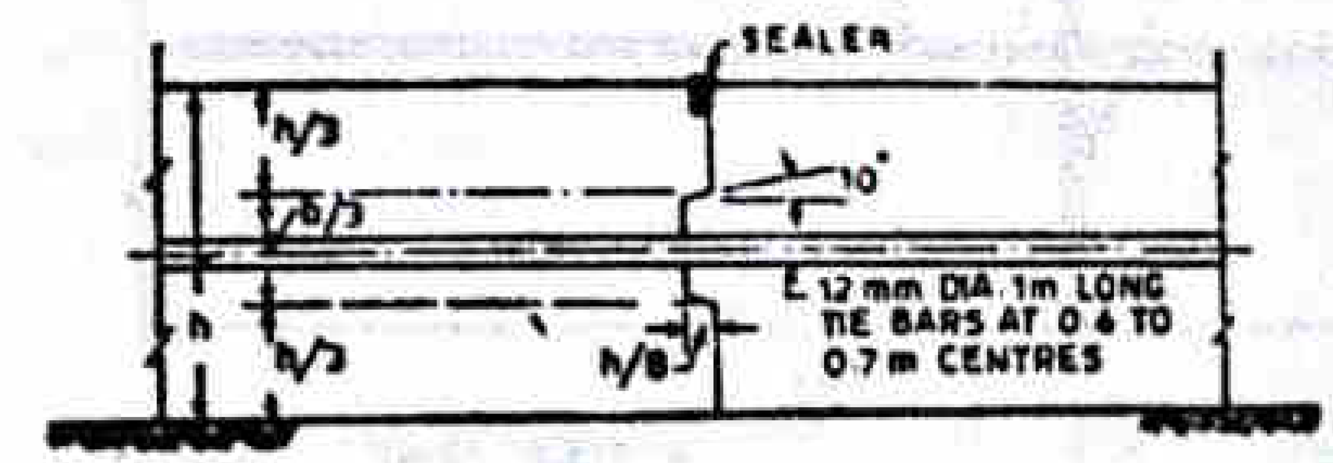
In cement concrete slab of thickness 20 cm, (a) 10 mm diameter deformed tie bars of length 35 cm or plain bars of length 45 cm are placed at 45 cm spacing, or (b) 12 mm diameter deformed bars of length 40 cm or plain bars of length 55 cm are placed at 64 cm spacing. In slabs of thickness 25 mm, tie bars of diameter 10, 12 or 14 mm and length 35 to 46 cm are placed at 30 to 62 cm spacing.



(a) Plain Butt Joint



(b) Butt Joint with Tie Bar



(c) Tongue and Groove Warping Joints

Fig. 8.22 Longitudinal Joints

Arrangement of joints

The joints in transverse direction are placed as follows :

(a) Staggered arrangement

(b) Uniform arrangement

(c) Skew arrangement

Figure 8.23 shows the arrangement of joints.

It is observed that where transverse joints are placed, staggered on either side of the longitudinal joints as shown in Fig. 8.23 (a), *sympathetic cracks* are often formed in line with the transverse joints. It is therefore recommended to provide joints across the longitudinal joint in same transverse alignment as shown in Fig. 8.23 (b).

It is always attempted to avoid the skew alignment of the joints but in some typical layout at intersections, it may be required to provide skew arrangement. At places where skew arrangement cannot be avoided, the acute corners so formed are strengthened with reinforcement as shown in Fig. 8.23.

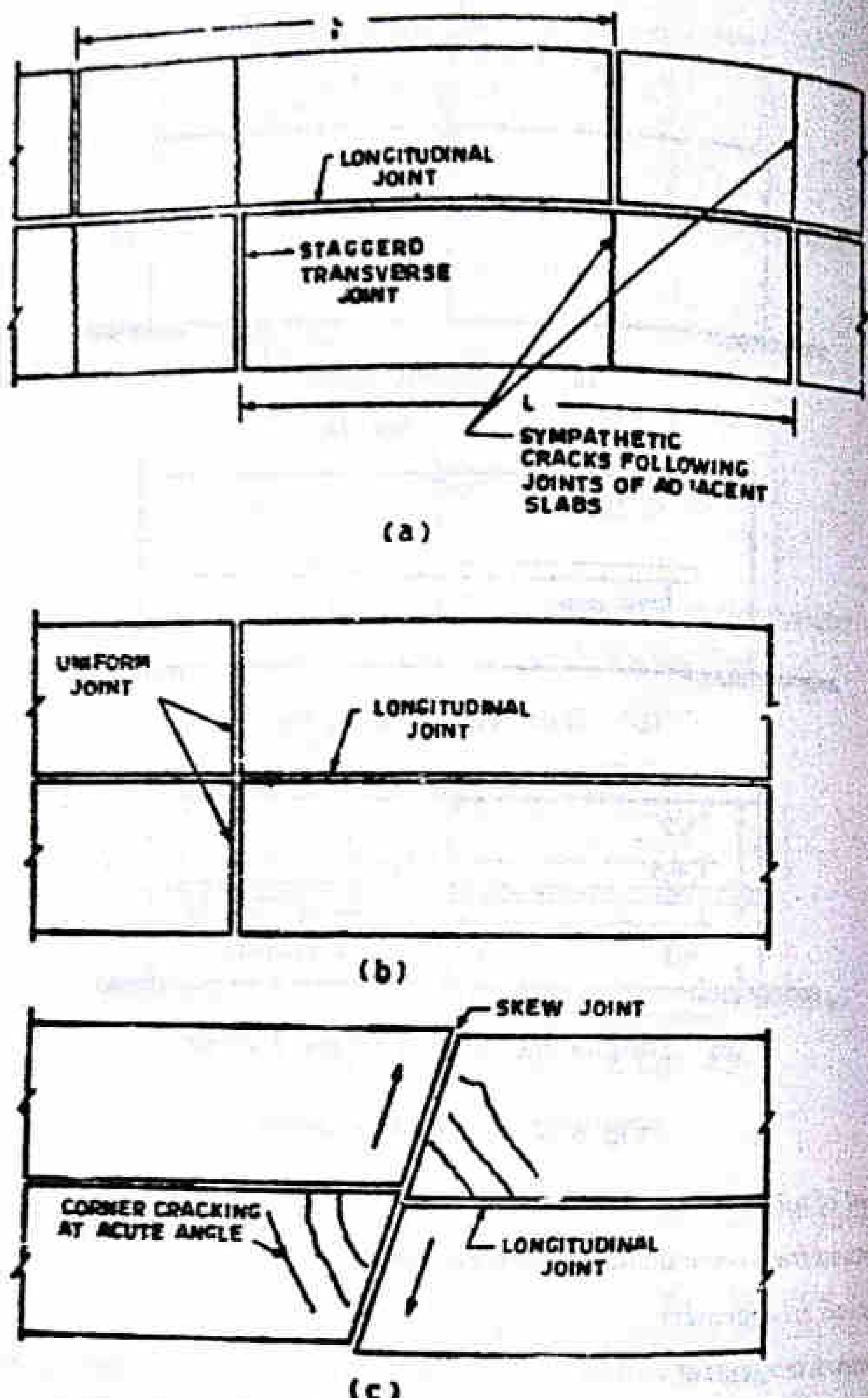


Fig. 8.23 Arrangement of Joints

8.9 JOINT FILLER AND SEALER

Joints form the break in the cement concrete pavement and these can allow the infiltration of water and ingress of stone grits. The infiltration of water damages the soil subgrade and gives rise to the phenomenon known as *mud pumping* especially if the subgrade is of clayey soil. If stone grit enters into the joint space, the effective joint width gets reduced and faults like *spalling* of joint edges take place. In extreme cases the *blowups* take place. Thus the joint spaces are first filled with compressible filler materials and the top of the joints are sealed using a sealer.

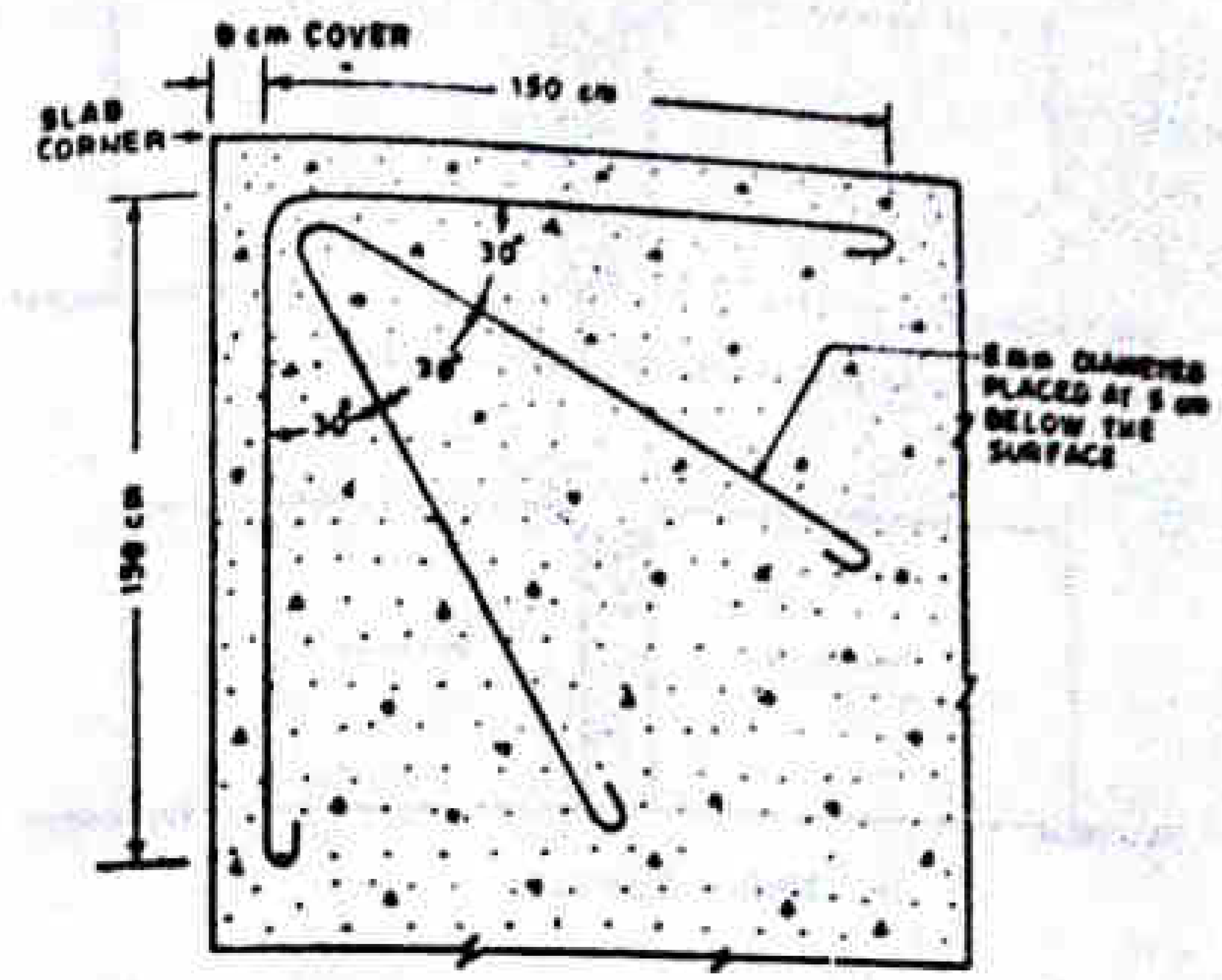


Fig. 8.24 Strengthening of Corner Region

8.9.1 Joint Filler

Joint filler should possess the following properties :

(a) Compressibility

(b) Elasticity

(c) Durability

Thus the filler material should be compressible and elastic i.e., it should be able to get compressed and on release of the compression, should be able to regain its shape and be elastic.

Figure 8.25 explains the functioning of the filler during changes in seasons. The filler is placed during construction as in Fig. 8.25a and when the summer approaches, the pavement expands and the joint gap thereby reduces as shown in Fig. 8.25b; at this time, the filler is compressed. During winter that follows in a cycle, the slab edges move back and if the filler is inelastic, there will be formation of gaps as shown in Fig. 8.25c. These gaps are detrimental and in fact render the joint as with a gap.

Type of Joint Filler

Following are the few types of joints filler materials :

(a) Soft wood

(b) Impregnated fibre board

(c) Cork or cork bound with bitumen

Thus pre-formed fillers are made from fibres of soft board, fibre board, coir fibre or cork. It is required that the preformed strips of these materials are properly bonded together with bitumen. The bitumen content specified by IRC is 35 percent by weight.

Various properties required for the satisfactory filler materials are as under. These are based on IRC recommendations :

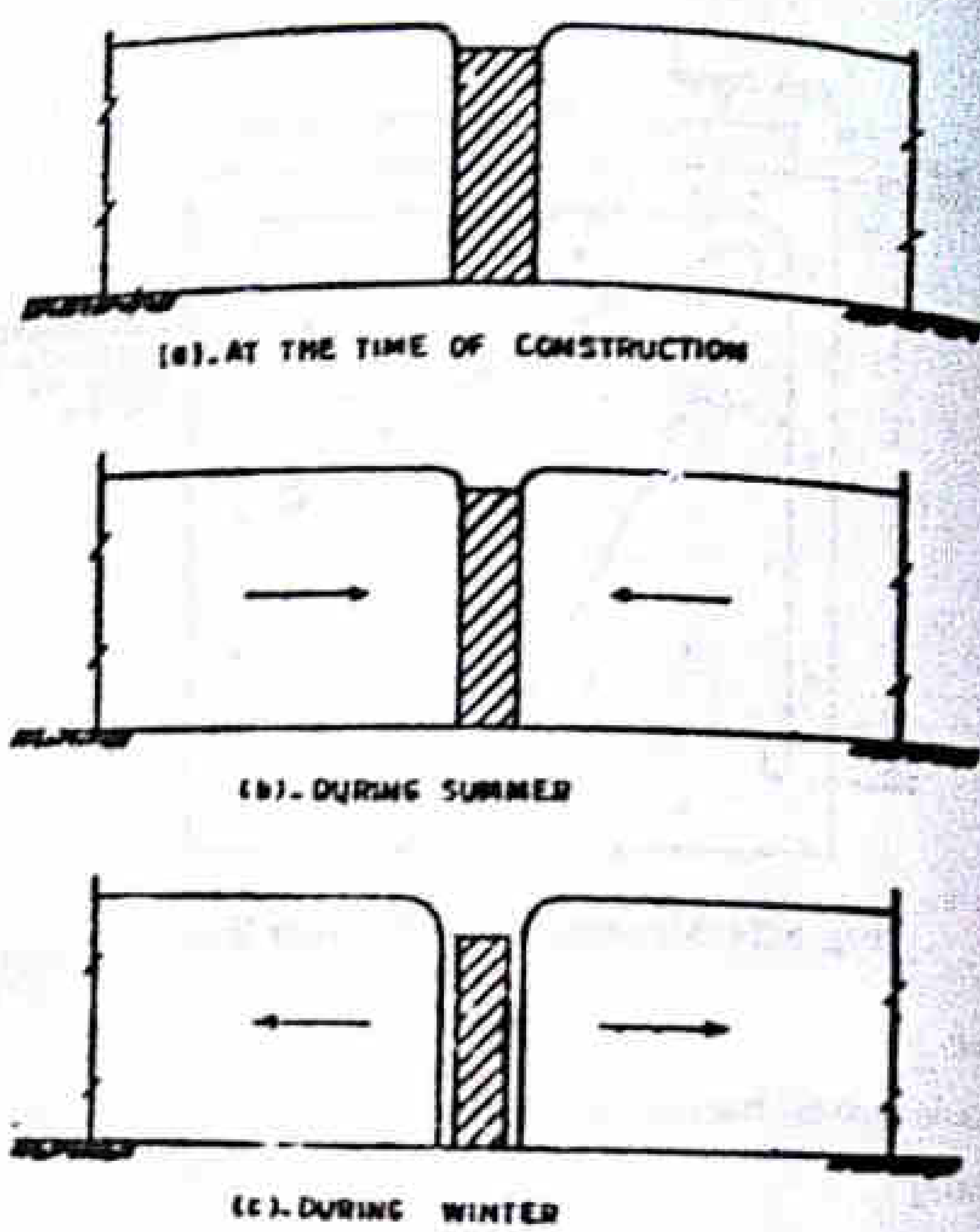


Fig. 8.25 Functioning of Joint Filler

(i) *Compression.* The pressure required to compress the specimen to 50 percent of its original thickness should be between 7 to 53 kg/cm^2. Materials should not show loss of its original weight by more than 3 percent.

(ii) *Recovery.* Specimen should recover at least 70 percent of original thickness after the load is applied and released after one hour at the end of third application of load.

(iii) *Extrusion.* The extrusion of one edge should not be more than 6.5 mm when the specimen is compressed to 50 percent of its thickness with three edges restrained.

The specimen should not show disintegration of fibre when tested. Non-extruding and resilient type bitumen-impregnated fibre may be preferred as joint filler material.

8.9.2 Joint Sealer

For effective sealing of joint for a long period, it is essential that the sealing compounds possess the following properties :

(a) Adhesion to cement concrete edges
(b) Extensibility without fracture
(c) Resistance to ingress of grit
(d) Durability

Different types of sealing compounds are in use. Bitumen is used either along or with mineral filler as a sealing compound. Rubber-bitumen compounds are also used for the purpose. Air-blown bitumens may be used with advantage, as they are less susceptible to the temperature changes.

The functioning of sealer is explained through Fig. 8.26. The proper positioning of sealer is shown as in Fig. 8.26a. As the winter approaches, the slab edges move apart causing an extension in the sealer material. At this instance the sealer forms a thin film and depending on its extensibility, either it maintains its continuity or it breaks. Once the sealer breaks the chains of maintenance, problems show up at the joints or slab edges. See Fig. 8.26b. Contrary to this, in summer depending upon the flow characteristics of the sealer material the bitumen would flow and spread around the joint to spoil the appearance of the cement concrete pavement and also decrease the smooth riding quality of the surface. This will render the film thickness unsuitable for the subsequent cyclic variations during winter. See Fig. 8.26c.

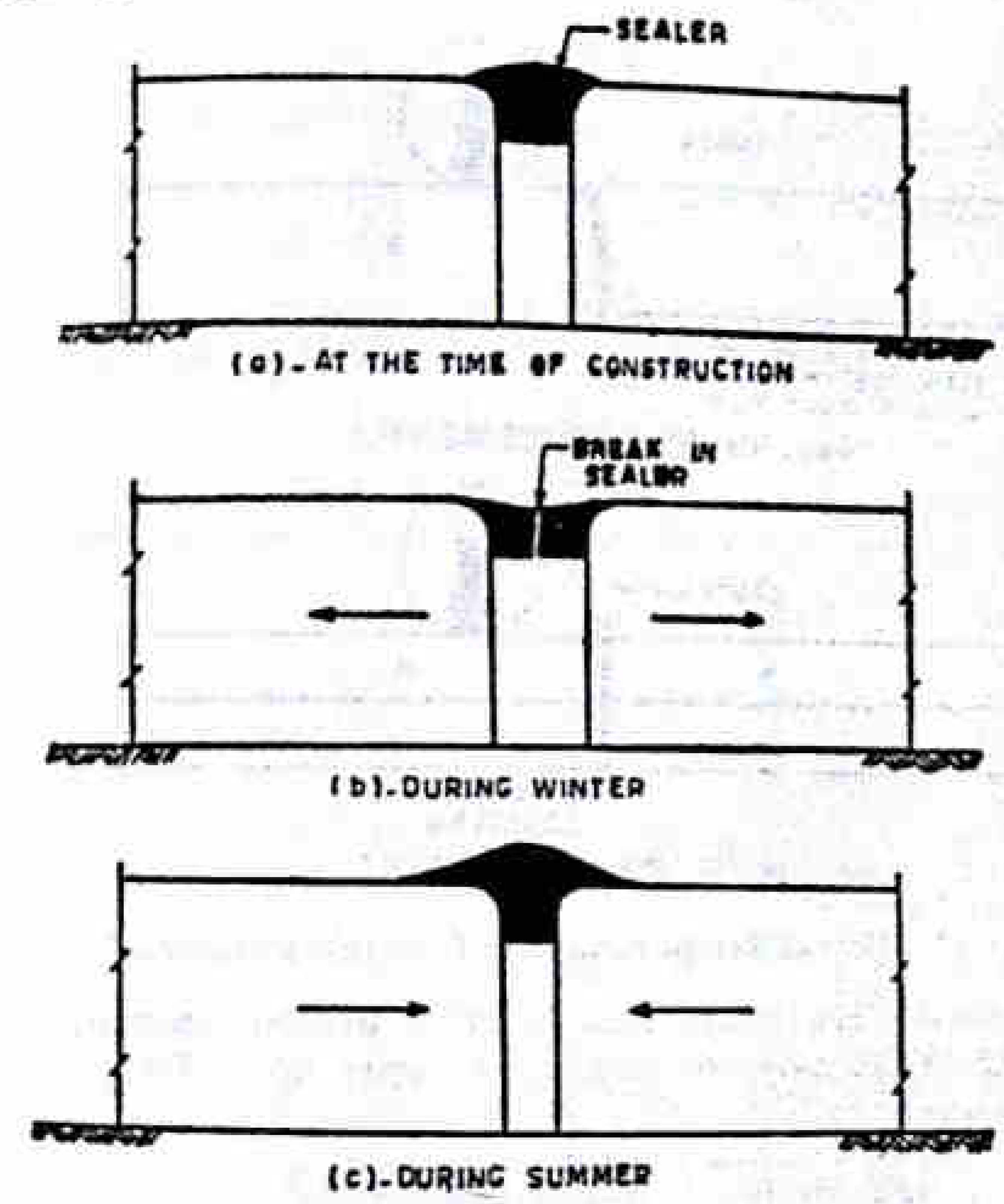


Fig. 8.26 Functioning of Joint Sealer

The sealing compound should be impermeable and be flexible to accommodate the slab movements; the compound should not flow in hot season or become brittle in cold season.

This shows that there are definite requirements of certain characteristics of the sealing compounds. IRC recommends the following requirements.

Characteristics	Requirements
Pour point	180°C max.
Softening point	75°C min.
Increase in softening point after heating to 20°C above pour point for 3hrs	5°C max.
Penetration value	15 to 50
Extensibility	6 mm max.
Resistance to *girt penetration (on impact test at 35°C in tenths of mm)*	20 max.

8.10 REINFORCED CONCRETE PAVEMENTS

The reinforcement used in cement concrete pavement is in the form of welded wire fabric or bar mats. The function of reinforcement is to hold the cracked slab portions together and thus not to allow them to open up any more. Sufficient load transfer across the crack is affected due to the reinforcement. See Fig. 8.27. It may be clearly understood that the quantity of reinforcement recommended for use, does not provide any added flexural strength to the pavement. The function of the reinforcement is more or less similar to the dowel bars in contraction joints and tie bars to withstand tensile forces due to the movement of slabs and the frictional resistance between the bottom fibre of the concrete and the subgrade.

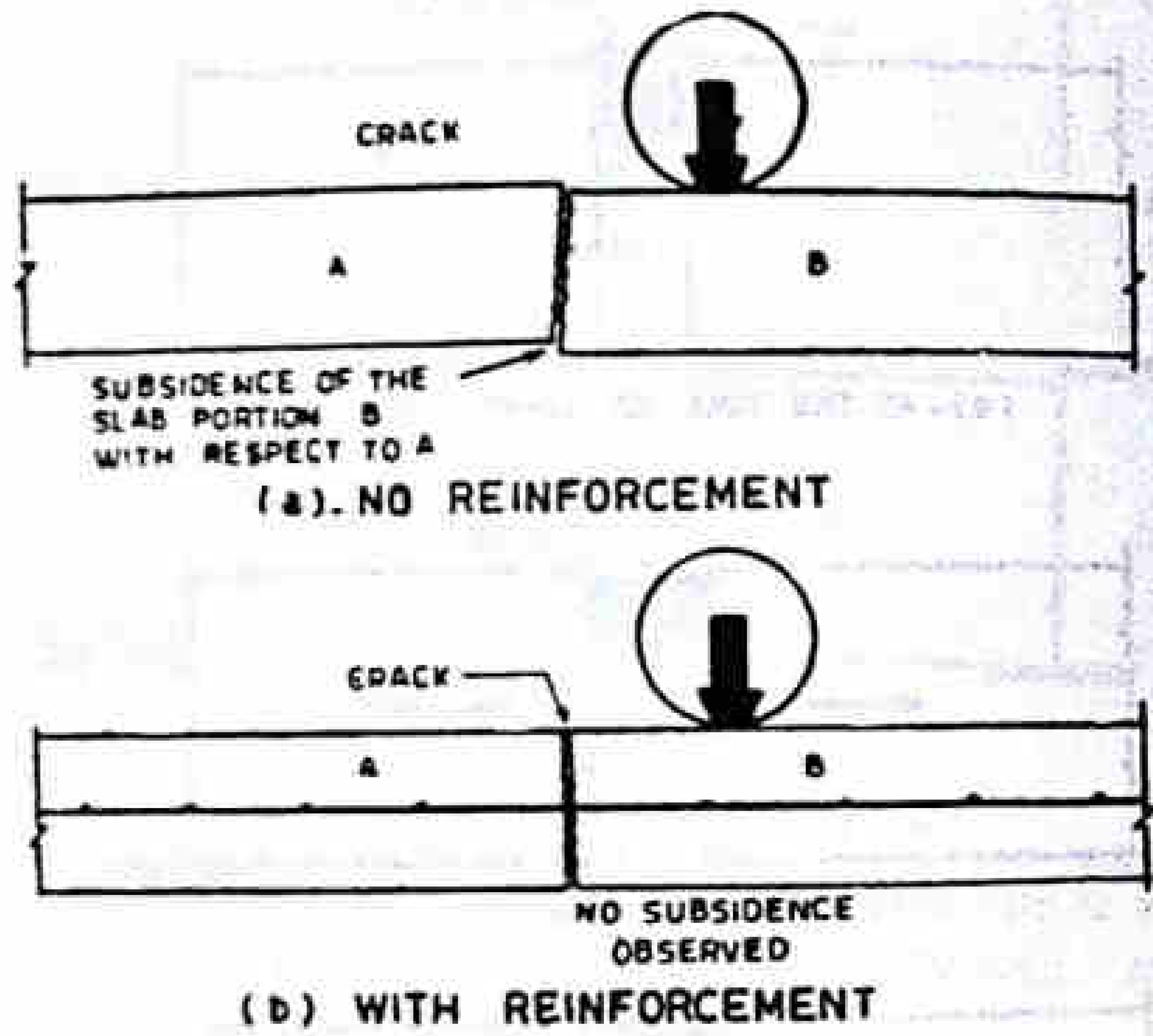


Fig. 8.27 Effect of Reinforcement in Concrete Pavement

Experience shows that hair cracks often occur at frequent intervals in R.C.C. pavement, but such cracks show no tendency to open up. The reinforcement recommended is as under :

Traffic intensity	Reinforcement, kg/m^2
Moderate	3.5
Heavy	7.0

In order to obtain the maximum advantage, it is suggested that greater quantity of the reinforcement should be placed in the longitudinal direction. Further the reinforcement should either be placed in the mid-depth or towards the top of the pavement for better functioning. Steel wire fabric should conform to IS : 1556 – 1960 and bar mats should conform to IS : 432 – 1960.

8.11 PRESTRESSED CONCRETE PAVEMENTS

The prestressing technique has been applied to the highway pavements in recent years. The prestressed pavement can be built in continuous length upto 120 m without joints. Elimination of the joints without inducing cracks in the pavement could be considered

advantageous, in view of the maintenance problems associated with the joints. To accommodate higher loads, there is obvious tendency of increasing the thickness. It may be realized that an increase in the thickness gives rise to a greater temperature differential of the slab and also greater frictional resistance. A thick slab is therefore undesirable as well as costly. By providing a residual compressive stress to the slab by use of tendons etc., the total tensile stress can fairly be neutralized and thus same unit thickness of prestressed concrete pavement could support heavier loads than plain concrete pavement and can be built for longer without joint.

Several investigators have contributed in the design of prestressed concrete pavements. The works of *Leonhardt* and *Morice* are noteworthy.

Following are few observations for the design :

(a) *Length* : A length upto about 120 m can be prestressed for the pavement.

(b) *Width* : A width of 3.6 m for prestressed pavement is desirable and a longitudinal joint therefore should be provided.

(c) *Thickness* : Because of the need to provide a required cover for tendons, the minimum recommended thickness is 15 cm.

(d) *Stress magnitudes* : A minimum value of 22 kg/cm^2 of prestress is recommended for 120 metre long prestressed pavement slabs 10 percent increase over this may be required to allow for losses in prestressing. A transverse prestress if required should be of 3 to 4 kg/cm^2.

Prestressing is applied either by pre-tensioning or by post-tensioning. For highway pavement, post-tensioning system has been used. Known methods include *Freyssinet*, *Magnel-Blaton Lee-McCall* and *Gifford-Udall* systems. Most of these systems employ wire of 7.0 mm diameter with ultimate tensile strength of 142-173 kg/cm^2. *Freyssinet* in France has utilised flat hydraulic jacks instead of steel tendons for highway pavements.

The construction of prestressed concrete pavements is difficult job and needs a skilled team. Due to the long length of tendons, there is a great amount of energy stored in it and any failure of anchor could be very severe. The Loss in prestressing due to subgrade restraint is one of the major problems in prestressed concrete pavement construction.

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PROBLEMS

1. List different methods of roads construction. Discuss their advantages and limitations.
2. Discuss the methods of earth excavation for road construction and the users of various equipment.
3. What are the problems in the construction of high embankments over weak foundation soils ? How are the various problems dealt with ?
4. Explain the objects of compaction and the effect of inadequate compaction. Discuss the advantages and applications of various compacting equipment for construction of subgrade and embankments.
5. Enumerate the steps for the preparation of subgrade.
6. Explain briefly the construction of earth roads. Discuss the advantages and limitations of earth roads.
7. Briefly list the method of construction of gravel roads.
8. Specify the materials required for construction of WBM roads. What are the uses and limitations of this type of road ?
9. Write down the construction steps for water bound macadam road.
10. What are the various types of bituminous construction in use ? Discuss the advantages and limitations of each.
11. Write short notes on :

    (a) Surface dressing

    (b) Interface treatment

    (c) Seal coat

    (d) Penetration macadam (Grouted macadam)

    (e) Bitumen bound macadam

    (f) Bituminous concrete

    (g) Sheet asphalt (Rolled asphalt)

    (h) Mastic asphalt
12. Compare the following methods of bituminous road construction :

    (a) Central plant mix and road mix methods.

    (b) Hot mix and cold mix

13. What are the materials required, plants and equipment and construction steps for the following methods of bituminous constructions :

    (a) Surface dressing

    (b) Penetration or grouted macadam

    (c) Bitumen bound macadam

    (d) Bituminous carpet

    (e) Bituminous concrete

14. What are the advantages and drawbacks of cement concrete roads ? Explain cement grouted and rolled concrete layers and their uses.

15. (a) Compare alternate bay and continuous bay methods of construction of cement concrete roads.

    (b) With the aid of neat sketches show the different types of joints and their positions.

16. What are the requirements of materials; plants and equipment for cement concrete road construction ? Discuss briefly.

17. Enumerate the steps in the construction of cement concrete pavement.

18. Discuss the object of the following types of joints; draw neat sketches :

    (a) Expansion joints

    (b) Contraction joints

    (c) Warping joints

    (d) Construction joints

    (e) Longitudinal joints

19. Write short notes on :

    (a) Thickened edge sections

    (b) Load transfer device

    (c) Sympathetic cracks

    (d) Strengthening corner of slabs

    (e) Prestressed concrete pavements.

20. Explain with sketches the requirements of joints filler and sealer. Discuss the desirable properties and the various materials in use.

21. Discuss the uses and limitations of reinforced cement concrete and prestressed concrete pavements for highways.

Chapter 9

Soil Stabilized Roads

9.1 INTRODUCTION

9.1.1 Objects

In developing countries like India the biggest handicap to provide a complete net work of road system is the limited finances available to build road by the conventional methods. Therefore there is a need to resort to one of the suitable methods of low cost road construction, followed by a process of stage development of the roads, to meet the growing needs of the road traffic. Thus apart from affecting economy in the initial construction cost of lower layers of the pavement such as sub-base course it should be possible to upgrade the low cost roads to higher specification at a later date without involving appreciable wastage, utilizing the principle of pavement construction in stages.

The construction cost can be considerably decreased by selecting local materials including local soils for the construction of the lower layers of the pavement such as the sub-base course. If the stability of the local soil is not adequate for supporting wheel loads, the properties are improved by *soil stabilization* techniques. Thus the principle of soil stabilized road construction involves the effective utilization of local soils and other suitable stabilizing agents.

9.1.2 Mechanics of Soil Stabilization

The term soil stabilization means the improvement of the stability or bearing power of the soil by the use of controlled compaction; proportioning and or the addition of suitable admixture or stabilizers. Soil stabilization deals with physical physico-chemical and chemical methods to make the stabilized soil serve its purpose as pavement component material. The basic principles in soil stabilization may be stated as :

(i) evaluating the properties of given soil.

(ii) deciding the method of supplementing the lacking property by the effective and economical method of stabilization.

(iii) designing the stabilized soil mix for intended stability and durability values.

(iv) considering the construction procedure by adequately compacting the stabilized layers.

Soil stabilization may result in any one or more of the following changes :

(a) Increase in stability, change in the properties like density or swelling, change in physical characteristics.

(b) Change in chemical properties.

(c) Retaining and desired minimum strength by water proofing.

Based on the above principles, the various technique of soil stabilization may be grouped as follows :

(i) *Proportioning technique* : Various locally available soils and aggregates are mixed in suitable proportions and compacted to serve the desired objective. For example the stability of a fine grained soil can be improved by the addition of gravel and sand components in suitable proportions. Like wise the stability of a cohesionless sand may be improved by the addition of some cohesive soil.

(ii) *Cementing agents* : The strength of the stabilized soil can considerably be increased by the addition of cementing agents like portland cement, lime or lime-fly-ash. Bituminous materials also impart binding effect to non-cohesive soils.

(iii) *Modifying agents* : If the stabilizer added in small proportion could modify the undesirable properties of certain soils (such as high plasticity, swelling, etc.) making them more useful as construction material, such stabilizers may be called *modifiers*. The most common modifier used in the case of highly plastic soils is lime. Portland cement also acts as modifier in some cases.

(iv) *Water proofing agents* : A compacted soil mass which is normally stable enough may become weaker or softer by the ingress of water or when subjected to making conditions. If absorption of water can be stopped or retarded by means of some water proofing agent, it will be possible to make use of such materials with advantage. The most common method of water proofing is by the use of bituminous materials.

(v) *Water repelling agents* : Almost the same function as water-proofing agent may be performed by some water repelling or retarding agents like vinsol resin and other resinous materials.

(vi) *Water retaining agents* : Some non-cohesive soils have sufficient stability when the compacted layer possesses slight moisture content, but the soil may become loose and less stable when completely dried. In such cases use of materials with deliquescent properties, like *calcium chloride* is likely to be useful to retain some moisture to impart some apparent cohesion and thus retain the stability. This incidentally can also reduce the dust nuisance in unsurfaced roads.

(vii) *Heat treatment* : Thermal stabilization has different useful aspects as regards clayey soils. There are desirable such as reduction in swelling properties and heat treated soil may be used as a soft aggregate in mechanical soil stabilization or as a pozzolanic additive in soil-lime stabilization.

(viii) *Chemical stabilization* : There are several chemicals, which when added single or in combination, even in trace quantities (less than 0.5 percent by weight of soil) may impart useful changes in certain types of soils. However considerable investigation and care is needed before adopting any of the costly chemicals. Chemicals have also been successfully used as additives in soil-cement and soil-lime stabilization.

In all the above methods, adequate compaction of the stabilized layers is the most essential requirement.

9.1.3 Investigation for Soil Stabilized Road

The various field and laboratory investigations needed for soil stabilized road construction may be listed as :

(i) Investigation of route and alignment

(ii) Soil survey and field identification of soils

(iii) Survey for availability of materials and transportation

(iv) Laboratory tests on soils

(v) Soil classification

The soil survey is carried out and disturbed soil samples are collected from shallow depth after removing the top soil and organic matter, along the alignment and from the sides. Field identification tests are carried out to assess the general quality of the soil. The laboratory tests may be for physical, chemical and engineering properties of the soils. Identification and classification tests such as liquid limit, plastic limit and mechanical analysis may be carried out. Chemical tests, include determination of organic matter, total soluble salts, sulphate and pH value. A suitable test such as CBR test may be conducted on the soil sample. The other tests to estimate the engineering properties are soundness, strength and abrasion tests on aggregate component and determination of swelling property on the fine fraction of the soil.

Soil classification systems followed for soil stabilization works are the HRB or Revised PRA classification system and also the united soil classification. However classification of some of the soils in India such as *black cotton soil* could not be properly made. Hence some modifications in the above two classification systems have been suggested by the Central Road Research Institute, New Delhi.

9.1.4 Soil Stabilization Methods

The methods of soil stabilization which are in common use are :

(i) Mechanical soil stabilization

(ii) Soil-cement stabilization

(iii) Soil-lime stabilization

(iv) Soil-bitumen stabilization

These methods of stabilization have been explained in the subsequent articles.

9.2 MECHANICAL SOIL STABILIZATION

9.2.1 Principles and Applications

Correctly proportioned materials (aggregates and soils) when adequately compacted to get a mechanically stable layer, the method is called mechanical stabilization. Thus the two basic principles in this method of stabilization are :

(i) Proportioning

(ii) Compaction

If a granular soil containing negligible fines is mixed with a certain proportion of binder soil, it is possible to increase the stability. Similarly the stability of a fine grained soil could be considerably improved by mixing a suitable proportion of granular materials to get a suitable gradation.

Mechanical stabilization has been successfully applied for sub-base and base course constructions. It has also been used as surface course for low cost roads such as village roads when the traffic and rainfall are low.

9.2.2 Properties of Soil-Aggregate Mixtures

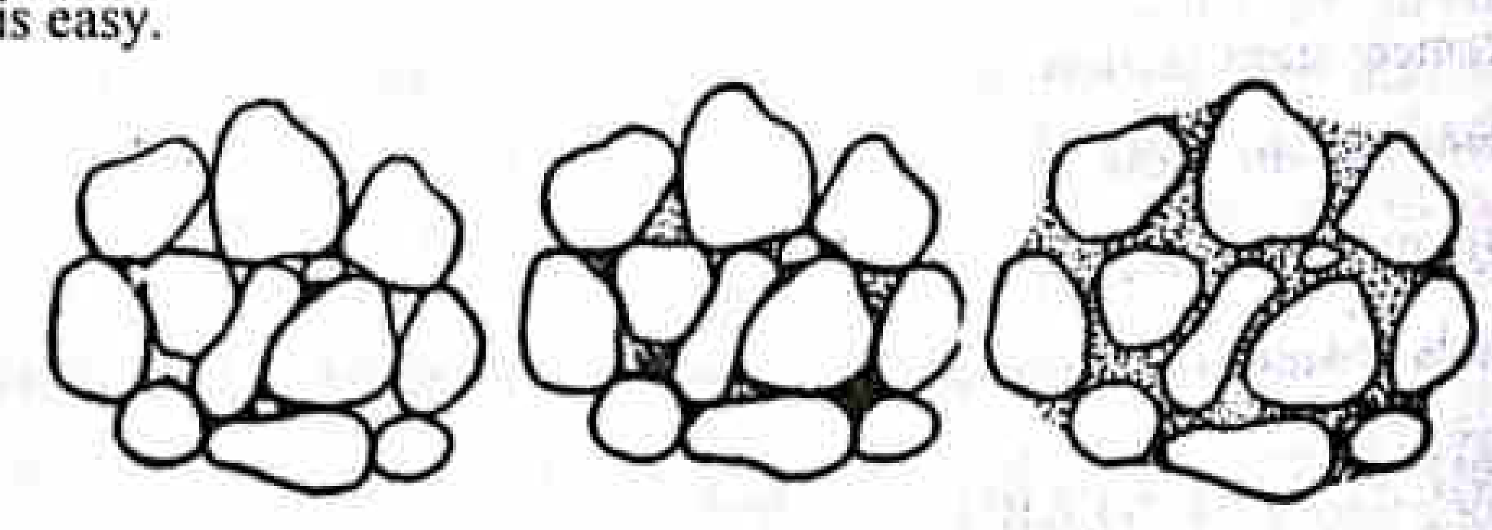
The desirable properties of soil-aggregates mixture are strength, incompressibility, less changes in volume and stability with variation in moisture content, good drainage, less frost susceptibility and ease of compaction. It is generally believed that the stability or a soil-aggregate mix could be increased by increasing its dry density. Hence proportioning of the mixes is done to attain maximum dry density.

Three typical states in which compacted soil-aggregate mixes could be formed are illustrated in Fig. 9.1.

Figure 9.1a shows the *state* when the aggregate is without fines. This obviously gives a mass which is stable only when confined (as there is no cohesion component), highly permeable, no frost action and practically no variation in volume or stability with moisture variations. It is not easy to compact such a granular material.

Figure 9.1 b shows the state when the voids in the compacted aggregate are just filled with compacted binder, thus retaining the frictional component of the mass due to the grain-to-grain contact of the aggregate. There is increased stability even when unconfined due to higher cohesion; but the mix is less permeable and is likely to cause frost action and variation in volume and stability due to moisture variations which depend much on the property of the binder.

Figure 9.1c illustrates the *states* when the aggregate is mixed with excess fines and compacted. As the aggregate grains have lost their contacts with each other and they *float* in the binder soil, the angle of friction and stability are decreased and mix is considered less desirable with poor drainage, more variation in stability and volume with moisture variation and high frost susceptibility. The mix being more workable, the compaction is easy.



(a) **Compacted aggregates with no fines** **(b)** **Compacted aggregates with compacted fines just filling the voids** **(c)** **Aggregate with excess fines**

Fig. 9.1 Typical States of Soil Aggregates

Thus it may be seen that the proportioning of the mix affects the properties considerably. With proper proportioning it is possible to attain a mix with best combination of the desirable properties.

9.2.3 Factors Affecting Mechanical Stability

The stability of mechanical stabilized soil mixes depend on the following factors :

(i) Mechanical strength of aggregates

(ii) Gradation

(iii) Properties of soil

(iv) Presence of salts, mica etc.

(v) Compaction

When the soil is used in small proportion to just fill up the voids in the aggregates, the crushing strength of aggregate is important. However, weak aggregates have also been successfully used in mechanical stabilization work. Grain size distribution of the combined mix would determine the maximum stability of the mix. A well graded aggregate-soil mix results in a mix with high dry density and stability values.

Properties of the soil such as plasticity characteristic would affect the performance of the mix. A mix containing soil with high plasticity index, results in poor stability under soaking conditions. Hence it is desirable to limit the plasticity index of the soil constituents. Salts like sulphates and presence of mica are also undesirable. But presence of salts like calcium chloride are considered beneficial. Compaction is one of the most important factors affecting the properties and performance of the stabilized layer. Effective compaction is desirable to produce high density and stability of the mix.

9.2.4 Mix Design in Mechanical Stabilization

The factors to be considered in the design of mix are gradation, density, index properties and stability. Of these, the graduation is the most important factor.

Gradation. The particle size distribution that gives maximum density is generally aimed at. The theoretical gradation for maximum density is given by :

$$P = 100\left(\frac{d}{D}\right)^n \qquad (9.1)$$

Here, P = percent finer than diameter 'd' (mm) in the material

D = diameter of largest particle, mm

n = gradation index, which have values ranging from 0.5 to 0.3 depending upon the shape.

Fuller assumed spherical particles while deriving the gradation equation for maximum density and the value of gradation index obtained for the spherical shape is, n = 0.5. In the case of particles which are angular flaky in shape, the value of gradation index n for maximum density will be less than 0.5, upto 0.3 or even lesser, depending upon the shape factor of the aggregates. A.S.T.M. and B.S.I. have proposed limits of particle size distribution for satisfactory base and surface courses.

Proportioning

When a few materials are available in the near vicinity, it is necessary to mix them in such a proportion which would produce a mix with highest density. As an example or coarse aggregate, sand and fine soil are available from three deposits or borrow pits, it is first essential to decide the best proportion of these component materials. Two graphical methods in common use for proportioning are the *triangular chart method* and *Rothfutch's method*.

Proportioning of Material by Triangular Chart

This method is applicable when three materials are to be mixed together to achieve a desired gradation (for maximum density). The materials may be conveniently divided as coarse aggregate, sand and fines (or silt and clay) and their percentages marked on a triangular chart as shown in Fig. 9.2. Suppose three materials A, B and C are available which are respectively coarse, medium and fine grained materials. Points A, B and C are plotted on the triangular chart knowing the grain size distribution (or proportion of coarse aggregate, sand and fines) of the three materials. Next step is to obtain the desired gradation D based on some gradation criterion or by using a gradation formula like that given in Eq. 9.1. The point D is also plotted in the triangular chart representing the desired gradation. Now the graphical construction for obtaining the proportions of A, B and C is made, by producing the line C D to meet the line A B at E. The proportions of coarse aggregate sand and fines are given by :

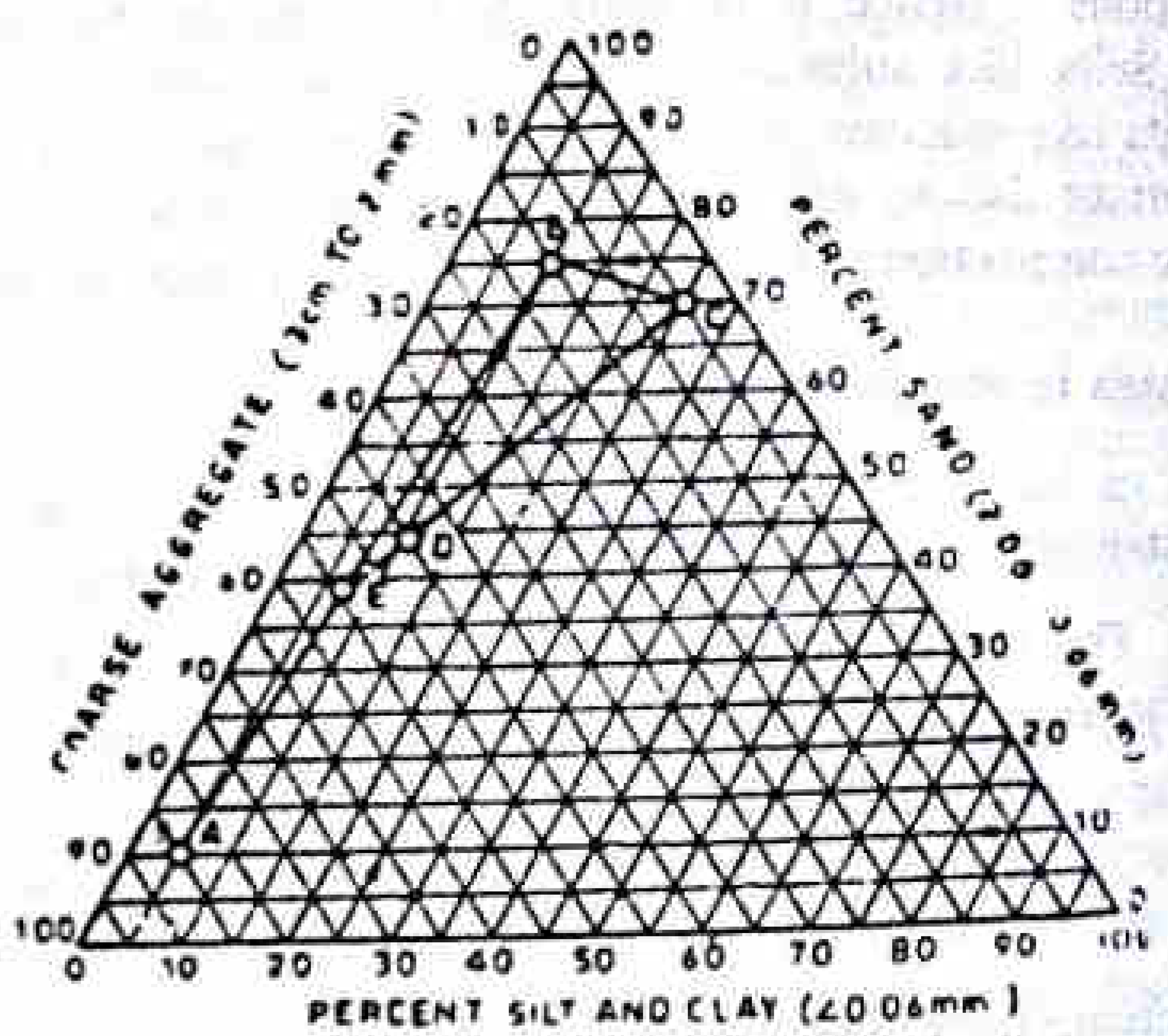


Fig. 9.2 Proportioning of Materials by Triangular Chart

$$\% \text{ coarse aggregate} = \frac{EB \times DC}{AB \times EC} \times 100$$

$$\% \text{ Sand} = \frac{AE \times DC}{AB \times EC} \times 100$$

$$\% \text{ Fines} = \frac{ED}{EC} \times 100$$

Proportioning of Materials by Rothfutch's Method

This method may be used when a number of materials have to be mixed together for obtaining a desired or design gradation. The desired gradation may be decided either based on recommended grain size distribution charts or tables or by using a theoretical equation like that given in Eq. 9.1.

On a graph paper with Y-axis representing percent passing and X-axis representing particle size (see Fig. 9.3) a diagonal line is drawn from point corresponding to (100 percent passing, maximum particle size of the material), to a point corresponding to (zero percent passing, smallest particle size of the materials). The different particle sizes are marked on X-axis corresponding to the mean values of percentage finer taken on the Y-axis, making use of the table of desired gradation or a suitable gradation equation. The particle size scale on X-axis is such that the desired gradation is straight line which was drawn first. Now on this chart the grain size distribution curves of the materials to be mixed are plotted. Suppose three materials A, B and C available locally are to be mixed, then the grain size distribution curves of these three materials are plotted as shown in Fig. 9.3 and the balancing straight lines of A, B and C are obtained, allowing only minimum of the areas on either sides of the balancing lines. The opposite ends of the balancing straight lines of A and B are joined, (i.e., zero percent passing of material A is joined with 100 percent passing of B). Similarly the opposite ends of balancing lines of B and C are joined. The points where these lines meet the desired gradation line (Fig. 9.3) represent the proportions in which the materials A, B and C are to be mixed. These values may be read from the Y-axis by projecting the points of intersections, as shown in Fig. 9.3.

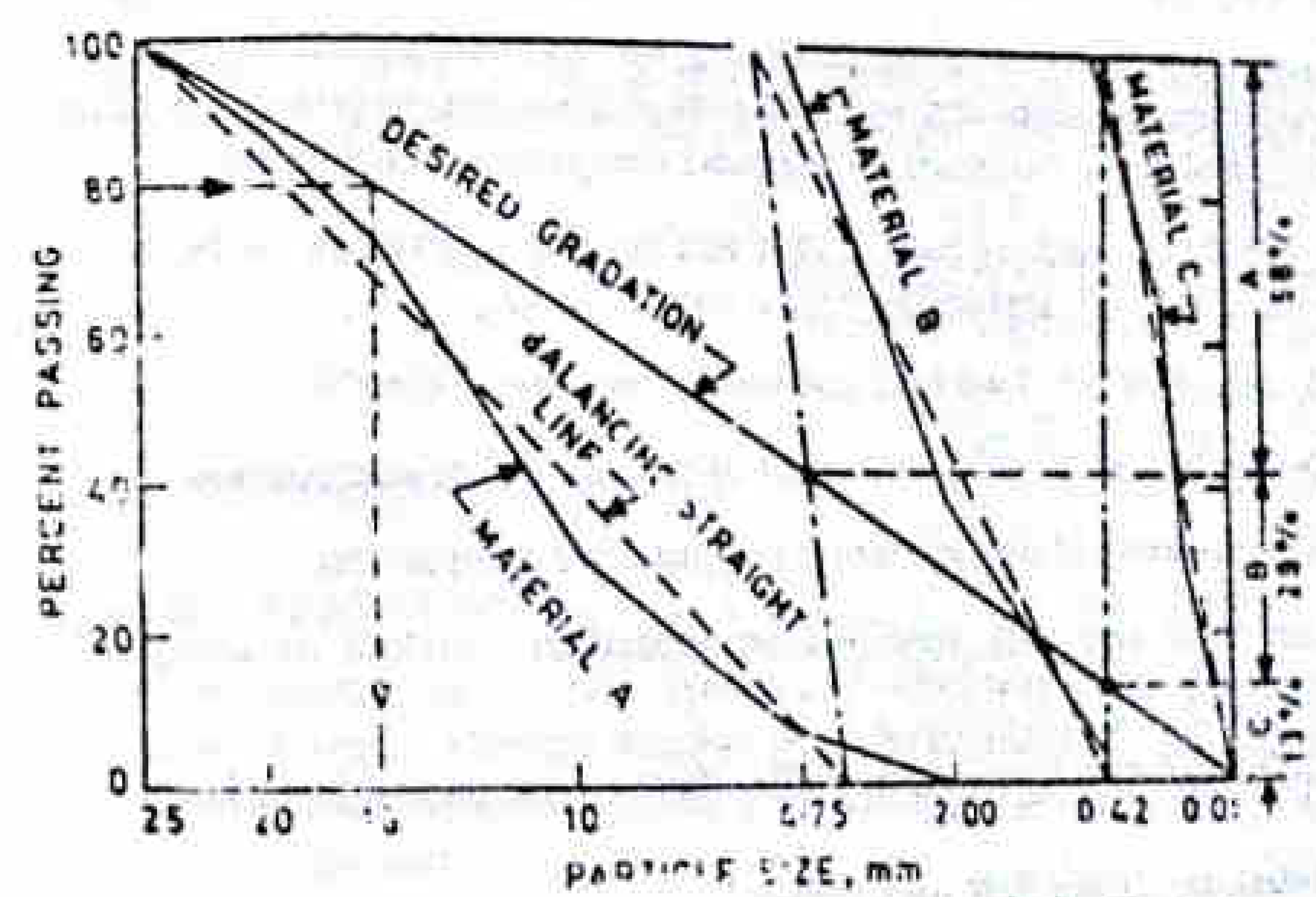


Fig. 9.3 Proportioning of Materials by Rothfutch Method

It is not always necessary that the proportion of the mix which gives maximum dry density should also give maximum stability, especially under soaked conditions. Hence it is also possible to design the proportions based on a stability test.

Density. Standard dry density is obtained in the laboratory compaction test. At least 95 percent of the standard dry density value is aimed at in the field compaction.

Index properties. The following are the recommended values of liquid limit and plasticity index for the material passing 0.425 mm sieve.

	Base course	Surface course
Liquid limit	25% max.	35% max
Plasticity index	6% max.	5 to 10 %

Stability. Any method of stability test may be adopted based on design, which is suitable for the type of material. Soaked C.B.R. test is commonly adopted.

9.2.5 Construction Procedure

Materials

The construction materials are collected from the selected borrow pits and are stacked along the sides of the road in the desired proportions.

Equipment

The equipment needed are for excavation upto shallow depth, haulage for short distance and for compaction. Machinery or manual labour may be used for excavation and haulage. For compaction, roller of the suitable type and weight is necessary depending on the materials to be compacted. Refer article 8.2.5 for choice of equipment.

Construction steps

(i) The subgrade is prepared as explained in Art. 8.2.4.

(ii) The materials are mixed to the desired proportions as per design. Generally the proportions are converted on volume basis.

(iii) The existing moisture content is checked by a rapid method and additional water required is spread and the material is re-mixed.

(iv) The wet mix is spread to the desired grade and compacted by rollers. Rolling is started from the edges and with adequate longitudinal overlap, it is continued up to centre. Rolling is continued till adequate compaction is achieved.

When two layers such as base course and surface course are to be constructed, the process is repeated with appropriate proportions of mixes.

(v) *Field control tests* : Two field control tests necessary are :

(a) determination of moisture content of the mix before compaction.

(b) determination of density during and just after compaction.

Based on these two tests, modification needed in moisture or compaction may be decided.

(vi) The stabilized road is opened to traffic after the compacted layer hardens by drying.

9.2.6 Stabilization Using Soft Aggregates

When hard variety of aggregates are not locally available, the local soft aggregates may have to be used for construction in order to keep the construction cost as low as possible. The soft aggregate have low crushing strength and low aggregate impact value. Still they have been successfully adopted in the construction of mechanically stabilized sub-base course and even in base and surface course layers. The common soft aggregates used in road construction are kankar, moorum laterite and broken brick aggregates. Because of the low strength, the aggregates are likely to break down at their points of contact. If the aggregate is mixed with more proportion soil so that each particle of soil aggregate is enveloped by soil as shown in Fig. 9.1c, then there is no problem of crushing of these aggregates during compaction and under traffic loads. When the proportion of soft aggregates to soil is approximately 1 : 2, best results are obtained.

A method of stabilization using brick aggregates and soil was suggested by S. R. Mehra and therefore the method is popularly known as Mehra's method of stabilization. IRC has standardised the method of stabilisation using soft aggregates.

Mehra's method of stabilization. Base course material consists of compacted soil with sand content of size less than 0.425 mm and greater than 0.075 mm being not less than 50 percent and plasticity index 5 to 7.

Wearing course material consists of brick aggregates and soil is mixed in the ratio 1 : 2. The sand content of size less than 0.425 and greater than 0.075 mm in the soil should be not less than 33 percent and the plasticity index 9.5 to 12.5%. However when bituminous surface treatment is desired, the plasticity index is limited to 8 to 10%.

The Mehra's method of construction is briefly given here :

(i) Soil is collected from approved borrow pits and stacked on road side.

(ii) Water is added upto OMC, and soil is mixed and spread to a desired camber and grade.

(iii) 11½ cm thick loose base course material (sandy soil) is spread and rolled by 8 tonnes power roller to a compacted thickness of about 7½ cm.

(iv) Surface course material (brick aggregate + soil in the ratio 1 : 2) mixed with adequate water is spread to 11½ cm loose thickness and the layer is rolled by 8 tonnes power roller to a compacted thickness of about 7½ cm.

(v) After rolling, the surface is watered and left overnight. The surface is again rolled and finished.

(vi) The road is closed to traffic for four to five days and kept sprinkled with water. For next few days, only rubber tyred traffic is allowed and after about two weeks the road is opened to all traffic.

Mehra's method of construction can carry 50 tonnes per day of mixed traffic in places with light rainfall. With bituminous surface treatment, the road can given satisfactory service upto about 200 tonnes per day even in places with heavy rainfall.

9.3 SOIL-CEMENT STABILIZATION

9.3.1 Principles and Applications

Soil-cement is an intimate mix of soil, cement and water which is well compacted to form a strong base course. Cement treated or cement modified soil refers to the compacted mix when cement is used in small proportions to impart some strength or to modify the properties of the soil.

In granular soil, the mechanism of stabilization is due to the development of bond between the hydrated cement and the compacted soil particles at the points of contact. In fine grained soil, the stabilization is due to reduction in plasticity and formation of matrix enclosing small clay lumps. Degree of stabilization depends on nature of soil, proportion of cement compacting moisture cement and the dry density of the compacted mix. By the increasing percentage of cement added, there is an increase in the strength and durability of soil-cement and a decrease in volume change, moisture movement and plasticity.

Soil-cement can be used as a sub-base or base course of all types of pavements. However, as the material has poor resistance to abrasion and impact, this can not be used as a surface course. A bituminous wearing course is placed over the base course.

9.3.2 Factor Influencing Properties of Soil-Cement

Various factors which influence the properties of cement treated soils are soil, cement, pulverization and mixing compaction, curing and additives, if any.

Soil

The physical properties of the soil like particle size distribution, clay content, specific surface, liquid limit and plasticity index affect the properties of soil-cement. Also surface chemical factors, organic matter and sulphate content alter the properties and durability of soil-cement.

The cement content required for satisfactory stabilization of soils depends on the soil type. The cement requirement for a well graded granular soil would be much lower than that for a poorly graded or fine grained soil with high clay content. It has been generally observed that the cement requirement for satisfactory stabilization of soils increases with specific surface area of the soil.

The constituents of soils and the cation exchange capacity of soil influence the reaction with cement. The presence of certain types of organic matter in soil retard the rate of hardening of soil-cement and also cause reduction in strength. Sulphates present in soil and in ground water are in general detrimental to soil-cement stabilization. Magnesium sulphate may attack even the hydrated cement and cause reduction in strength and durability of the soil-cement.

Cement

An increase in cement content generally causes increase in strength and durability. Both normal and air entraining cement give almost the same results of stabilization.

Pulverisation and mixing

Better the pulverisation and degree of mixing, higher is the strength. Presence of unpulverised dry lumps of soil reduces strength and durability of soil-cement. However wet soil lump are not as harmful as dry lumps. The properties of soil cement could be considerably improved by pulverizing the soil such that the size of the lumps is as small as possible. Uniformity of mixing soil, cement and water is desirable. The degree of mixing is measured in terms of mixing efficiency which is the percentage of strength of the specimens prepared from field mix expressed in terms of the strength of specimens prepared from thoroughly remixed laboratory specimens. Increase in mixing period of wet mixing and delay in compaction after wet mixing cause reduction in density, stability and durability of the soil-cement.

Compaction

There is an optimum moisture content corresponding to maximum value of dry density or strength of a soil-cement mix. There is nothing like water cement ratio in soil-cement, as in cement concrete. The moisture that is added for adequate compaction is more than enough for the purpose of hydration of cement. If the dry density of the mix is increased by increasing the amount of compaction, the strength and durability of the soil-cement also increase.

Curing

During curing adequate moisture is to be retained. Higher temperature of curing accelerates the rate of gain in strength, the strength also increases with age.

Additives

There are various useful activities to soil-cement which improve the properties. Lime is a useful additive when clayey soil or an organic soil is to be stabilized. Sodium hydroxide, sodium carbonate and calcium chloride are some of the useful chemical additives to soil-cement.

9.3.3 Design of Soil-Cement Mix

There are various mix design methods, the most commonly known being the British methods and PCA method.

The British method of mix design is based on the compressive strength of specimens cured for 7 days. Soil-cement specimens are prepared with different cement contents in constant volume moulds of size 5 cm diameter and 10 cm height by compacting at OMC and Proctor density. The compressive strength of these specimens are tested after 7 days of curing and a graph is plotted with cement content versus compressive strength, kg/cm^2 as shown in Fig. 9.4. The cement content corresponding to a strength of 17.5 kg/cm^2 is taken as design cement content and this is considered quite adequate when the soil-cement is to be used for base course of highway pavements with light to medium traffic. However for heavy traffic, a higher strength factor of 28 to 35 kg/cm^2 may be adopted for mix design.

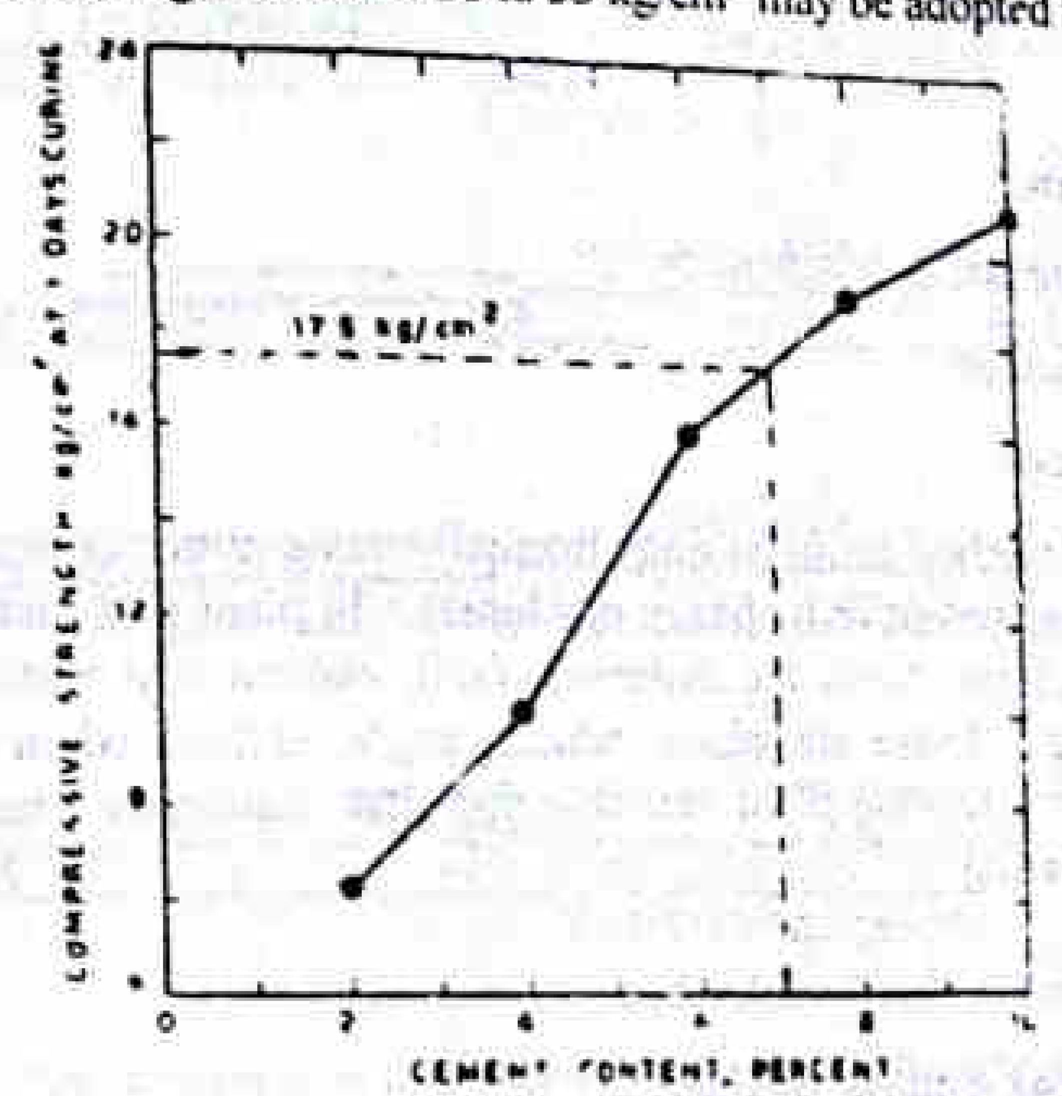


Fig. 9.4 Compressive Strength for Varying Cement Contents

In the P.C.A. method the design criteria are based on durability or ability of the soil-cement specimens to withstand the specified *wet-dry* cycles and *freez-thaw* cycles. Durability is decided based on resistance to loss in weight due to brushing the surface, volume change and moisture content during the specified durability cycles.

According to the PCA mix design criteria, the maximum soil-cement loss after the 12 cycles of wet-dry or freeze-thaw tests should not exceed the following limits.

Max. brushing loss, percent by weight	Soil type, HRB classification
14	A-1,A-3,A-2-4,A-2-5
10	A-2-6,A-2-7,A-4,A-5
7	A-6,A-7

The maximum volume change during the durability cycle should not exceed 2 percent of moulded volume of the specimens. The moisture contents during the durability tests should not exceed the saturation moisture content of moulded specimens.

The soil-cement mix designed by the above methods are generally considered to be strong and durable enough to replace an equivalent thickness of granular base course like WBM construction. However Highway Research Board (USA) recommended a reduced thickness of 75 percent of granular base course when soil-cement base course is constructed over weak subgrades. There is no accepted methods for thickness design of soil-cement base course.

9.3.4 Construction of Soil-cement Base Course

Materials

The soils to be stabilized, either from the site or from near-by borrow pits are collected and pulverized. From practical consideration, the following properties are recommended for the soils to be selected :

Passing 4.75 mm sieve	> 50 percent
Passing 0.075 mm sieve	< 50 percent
Liquid limit	< 40 percent
Plasticity Index	< 18 percent

Plants and Equipment

There are two methods of construction :

(i) Mix-in-place method

(ii) Plant mix method

The mix-in-place method could be done manually using minor equipment or by using simple agricultural equipment or by heavy machinery. In plant mix method large mixing plants are used which can batch the materials, (soil, cement and water) mix them and deliver for spreading. These are plants which can be shifted when the construction progresses, known as travelling plant and also there are stationary plants. Compacting equipment are also needed.

Construction Steps for Mix-in-Place Method

(i) Preparation of subgrade or sub-base

(ii) Pulverisation of soil

(iii) Application of cement and dry mixing

(iv) Addition of or spraying water and remixing

(v) Spreading and grading

(vi) Compaction

(vii) Curing-the soil-cement layer is allowed moist curing either by preventing the moisture to escape or by covering with moist soil.

(viii) Joint with old work

(ix) Field control tests.

In the case of plant mixing steps (ii), (iii) and (iv) are performed in the mixing plant.

The compaction is done with minimum possible delay after wet mixing. Smooth wheel rollers are most commonly adopted. During curing, efforts are made to retain as much moisture as possible out of the water added for compaction. Spraying of bituminous material or covering with moist soil are common methods of curing.

Joint with old work or construction joint is to be made carefully as the old soil cement is brittle and the edges may get severely damaged if specific care is not taken during scarifying, pulverizing, mixing and compaction of the new section of base course. Figure 9.5 illustrates the method of constructing joint between completed section of soil-cement base without damaging it.

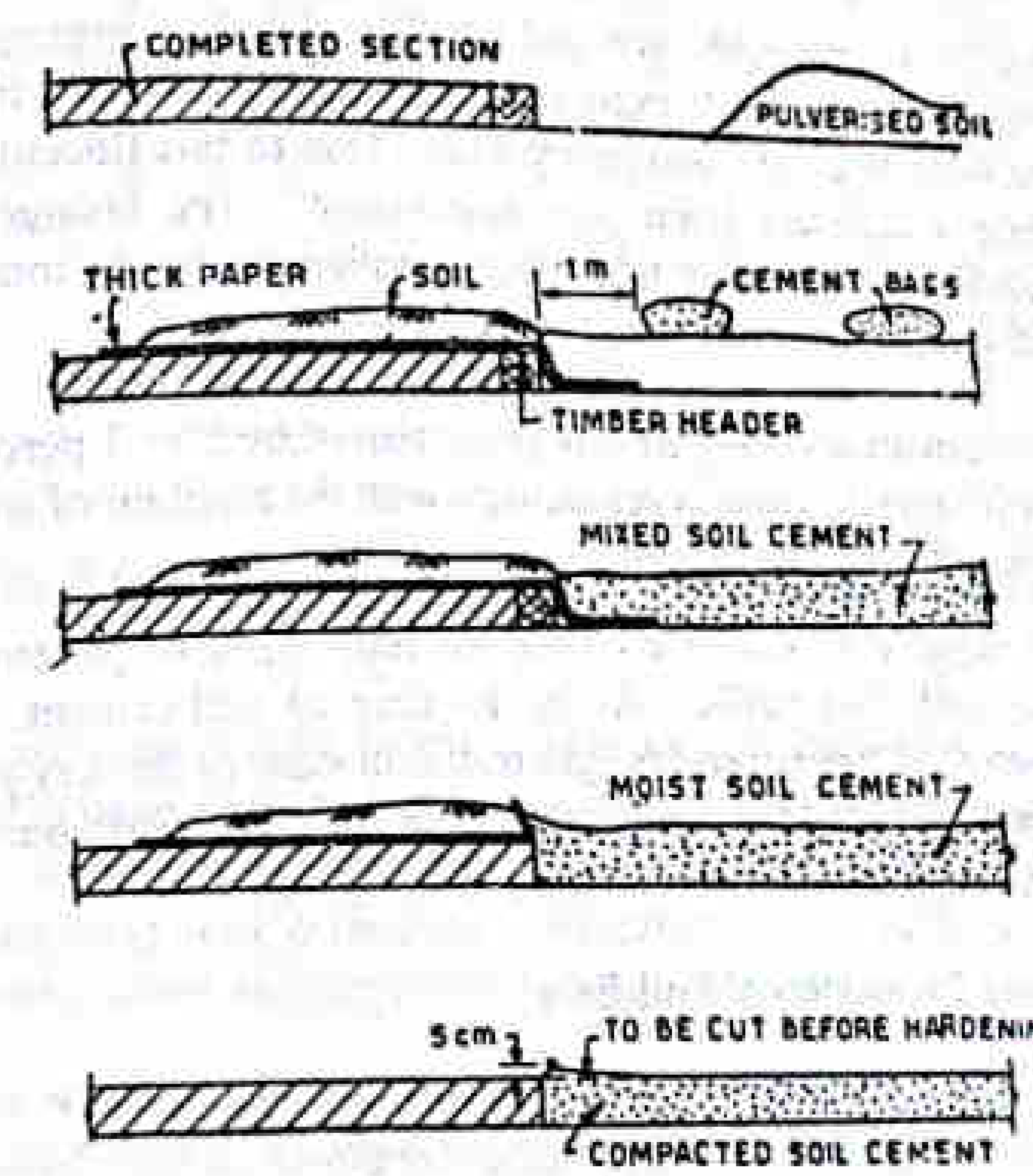


Fig. 9.5 Protection of Joint in Soil-Cement Construction

The various field control tests are :

(a) checking of moisture content of soil and soil-cement mix

(b) checking for degree of pulverization by sieving through 4.75 mm sieve

(c) testing of mixing efficiency by comparing the compressive strength of field mix and re-mixed and compacted specimens.

(d) checking cement content of mix.

(e) determining dry density of the compacted layer

(f) checking depth of the processed layer and surface regulating of finished layer.

9.4 SOIL-LIME STABILIZATION

9.4.1 Principles and Applications

Soil lime has been widely used either as a modifier for clayey soil or as a binder. In several cases both actions of lime may be observed. When clayey soils with high plasticity are treated with lime, the plasticity index is decreased and the soil becomes friable and easy to be pulverized, having less affinity with water. All these modifications are considered desirable for stabilization work. Lime also imparts some binding action even in granular soils. In fine grained soils there can also be *pozzolanic action* resulting in added strength.

When a clay is treated with lime, the various possible reactions are base exchange, coagulation or flocculation, reduction in thickness of water film around clay particles, cementing action and carbonation.

The fine clay particles react with lime and get flocculated or aggregated into larger particle groups which are fairly stable even under subsequent soaking. Plastic clay soils tend to agglomerate more than silty and sandy soils. Due to this flocculation; the lime-treated clays indicate a different grain size distribution. The changes in plasticity characteristics of soil-lime mixture also take place simultaneously; the total time required for the changes depending, on several factors including soil type.

The maximum dry density of soil-lime mix is decreased by 2 to 3 percent in terms of untreated soils; however this decrease in dry density with the addition of small proportion of lime does not cause reduction in strength.

Soil-lime is quite suitable as sub-base course for high types of pavements and base course for pavements with low traffic. As in the case of soil-cement, soil-lime also cannot be used as a surface course even for light traffic in view of its very poor resistance to abrasion and impact. Soil-lime is quite suitable in warm regions; but it is not very suitable under freezing temperatures.

9.4.2 Factors Affecting Properties of Soil-lime

The various factors on which the properties of soil-lime depend are soil type, lime content; compaction, curing and additives; if any.

Soil Type

Various soil properties affect the base exchange characteristics and pozzolanic action. The proportion of increase in strength in a soil-lime mix depends on the pozzolane in the soil.

Lime Content

Generally an increase in lime content causes a slight change in liquid limit and a considerable increase in plastic limit resulting in reduction in plasticity index. These are shown in Fig. 9.6 for a typical clayey soil. The rate of increase in plastic limit is first rapid and then the rate decreases beyond a certain lime content. This point is often termed *lime fixation point*. This is the approximate lime content that is considered to be used up for modification of clay. During this range the increase in stability of the clay-lime may not be noteworthy. When the lime content in the mix is further increased, there is a high rate of increase in stability. However when the lime content is increased beyond a certain proportion, the stability values generally start decreasing, as illustrated in Fig. 9.6.

With proper lime treatment it is possible to make the clay almost nonplastic with plasticity index reducing to practically zero. Increase in lime content also causes considerable reduction in swelling and increase in shrinkage limit. All these changes are desirable for stabilization of clay.

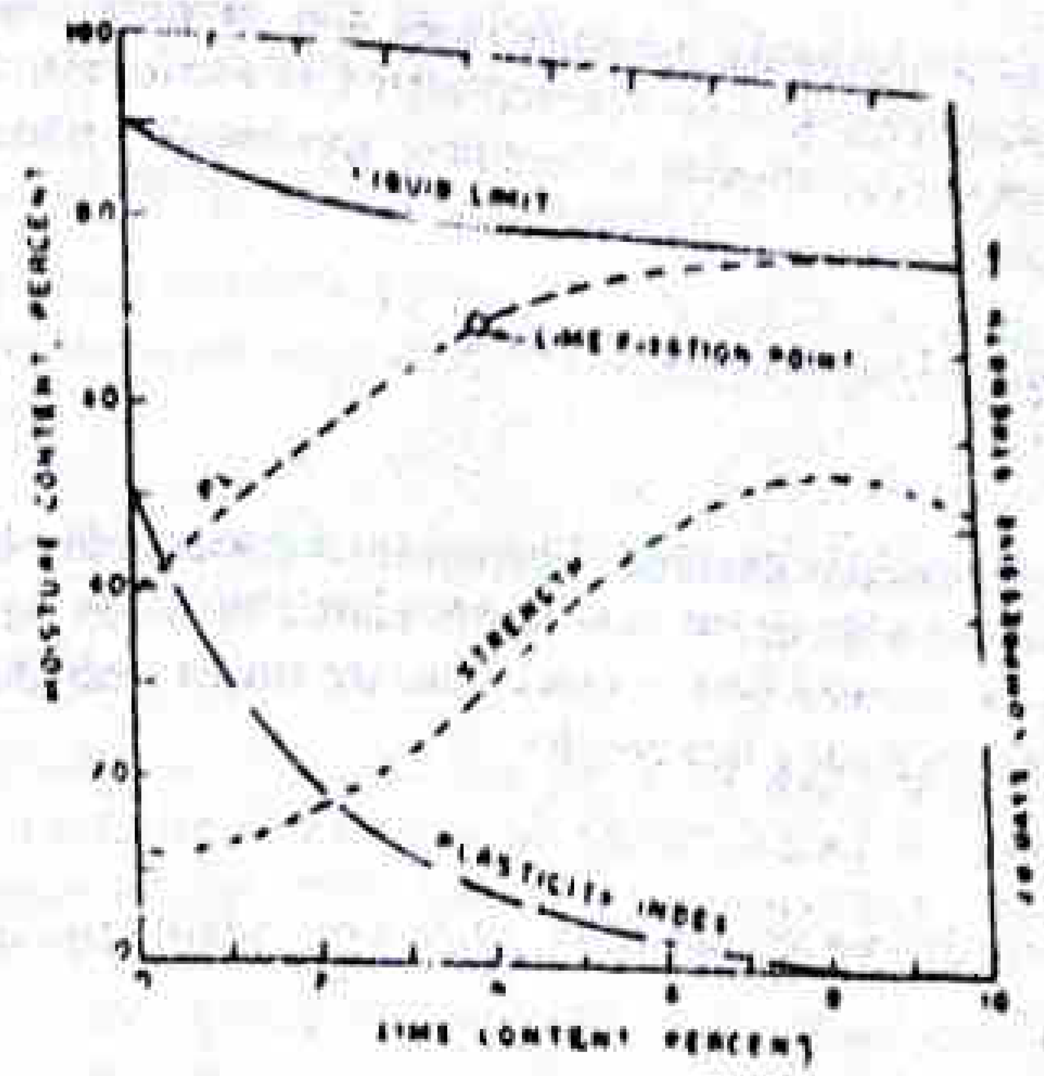


Fig. 9.6 Effect of Lime Content on Properties of Clay-lime

Types of Lime

After long curing periods all types of limes produce almost the same effects. However quick lime (CaO) have been found to be more effective than hydrated lime [$Ca(OH)_2$]. But quick lime has a tendency to cause skin burns of unprotected workmen, and hence need careful handling and protection. Hydrated lime is therefore commonly used in stabilization work, either as a dry powder or by mixing with water.

Compaction

The compacted density is important as regards the strength of soil-lime is concerned. Hence compaction is done at OMC and maximum density is aimed at.

Curing

The strength of soil-lime increases with curing period upto several years. The rate of increase in strength rapid during the initial period of curing, which also depends on the during temperature. At low temperature the rate of strength gain decreases considerably, below freezing point practically there is no gain in strength. The humidity of the surroundings during curing also affects the strength.

Additives

Addition of lime alone with soil often does not increase the strength of the mix as desired. Hence materials which increase the strength of soil-lime are tried as additives. Portland cement and pozzolanic materials like *flyash* and *surkhi* are most promising materials in this respect. Lime-flyash stabilization is cheap and is a methods with considerable scope for the construction of low cost roads in warm regions and where flyash is available as a waste product. Chemical additives like sodium metasilicate, sodium hydroxide and sodium sulphate are also found to be useful additives to soil-lime.

9.4.3 Design of soil-lime Mix

There is no standard method of mix design. If lime is used mainly as a modifier for highly plastic clay, then the lime content may be decided based on lime fixation limit or at a higher value to reduce the plasticity index and swelling values up to the desired limits. Some strength test also may be considered as a criterion for the design of mix. However the compressive strength of soil-lime specimens without additives, at seven days may be quite low.

9.4.4 Construction of Soil-lime Base Course

Materials

The soil to be stabilized is scarified. There is no recommended limit of test values for the soil to be stabilized with lime as even highly plastic soils can be suitably modified by lime. Fresh stock of hydrated lime or quick lime are stored near the site. It is desirable that the lime available is made a fine powder.

Plants and Equipment

The equipment needed are for scarifying, pulversing, mixing (in-situ) and compaction.

Construction Steps

(i) Preparation of subgrade.

(ii) Pulverisation of the soil to be stabilized.

(iii) Addition of part of lime as dry powder or as slurry with water and mixing.

(iv) Allowing the mixture to age for about a day or precond *toning* the soil, and re-mixing when pulverisation becomes easy.

(v) Adding rest of the lime, water if necessary and remixing.

(vi) Spreading to desired grade and compacting.

(vii) The soil-lime is protected from drying out and is allowed moist-curing.

(viii) Field control tests include checking moisture content at the time of compaction and checking dry density soon after compaction.

9.5 STABILIZATION OF SOIL USING BITUMINOUS MATERIALS

9.5.1 Principles and Application

The basic principles in bituminous stabilization are water proofing and binding. By water proofing the inherent strength and other properties of the soil could be retained. In case of the cohesionless soils the binding action is also important. Generally both binding and water proofing actions are provided to soil.

In granular soil the coarser grains may be individually coated and stuck together by a thin film of bituminous materials. But in fine grained soils bituminous material plugs up the voids between small soil clods, thus water proofing the compacted soil-bitumen.

Most commonly used materials are cutback and emulsion. As heating of large quantities of soil and bitumen is not possible, a suitable grade of cutback is chosen depending upon the climatic conditions and mixing problems. Emulsions also may be used, especially in places when there is scarcity of water for construction purposes. After the soil-bitumen (cut back or emulsion) is compacted, the layer is cured during which the water and volatiles evaporate and the mix hardens.

Bituminous stabilized layer may be used as a sub-base or base course of ordinary roads and even as surface course for roads with low traffic in low rain fall regions.

9.5.2 Factors Affecting Properties of Soil-bitumen

The various factors on which the properties of the stabilized mix depends are soil type, proportion of bituminous material, mixing, compaction, curing and additives, if any.

Soil

The particle size, shape and the gradation of the soil influence the properties of the soil-bitumen mix. A small proportion of fines in the soil is preferred, though high clay content is not desirable.

The relative affinity of soil particles for water and bitumen depends on surface chemical factors. Presence of cations with higher valency like calcium, potassium and aluminum have lesser affinity with water and hence are more desirable. Objectionable cationic charges in soil can be got exchanged with more desirable ions, by the use of chemical additives; for proper adhesion the soil surface and bitumen should have opposite electrostatic forces.

Types of Bituminous Material

Cutbacks of different grades give different stability values for a soil. The highest grade that can be mixed with the soil at the time of construction should be preferred. The type of cutback is also chosen depending on climatic conditions. Emulsion generally gives slightly inferior results than cutback.

Amount of Bitumen

Increasing proportion of bitumen causes a decrease in maximum dry density of soil-bitumen, but the stability increases and after a certain optimum bitumen content, it rapidly falls. Water absorption decreases with increase in bitumen content though a slight increase may be noted for very low bitumen content if the specimens are soaked for long period, such as 28 days. The variation of these properties with increase in bitumen content for a typical sandy soil has been illustrated in Fig. 9.7. The optimum bitumen

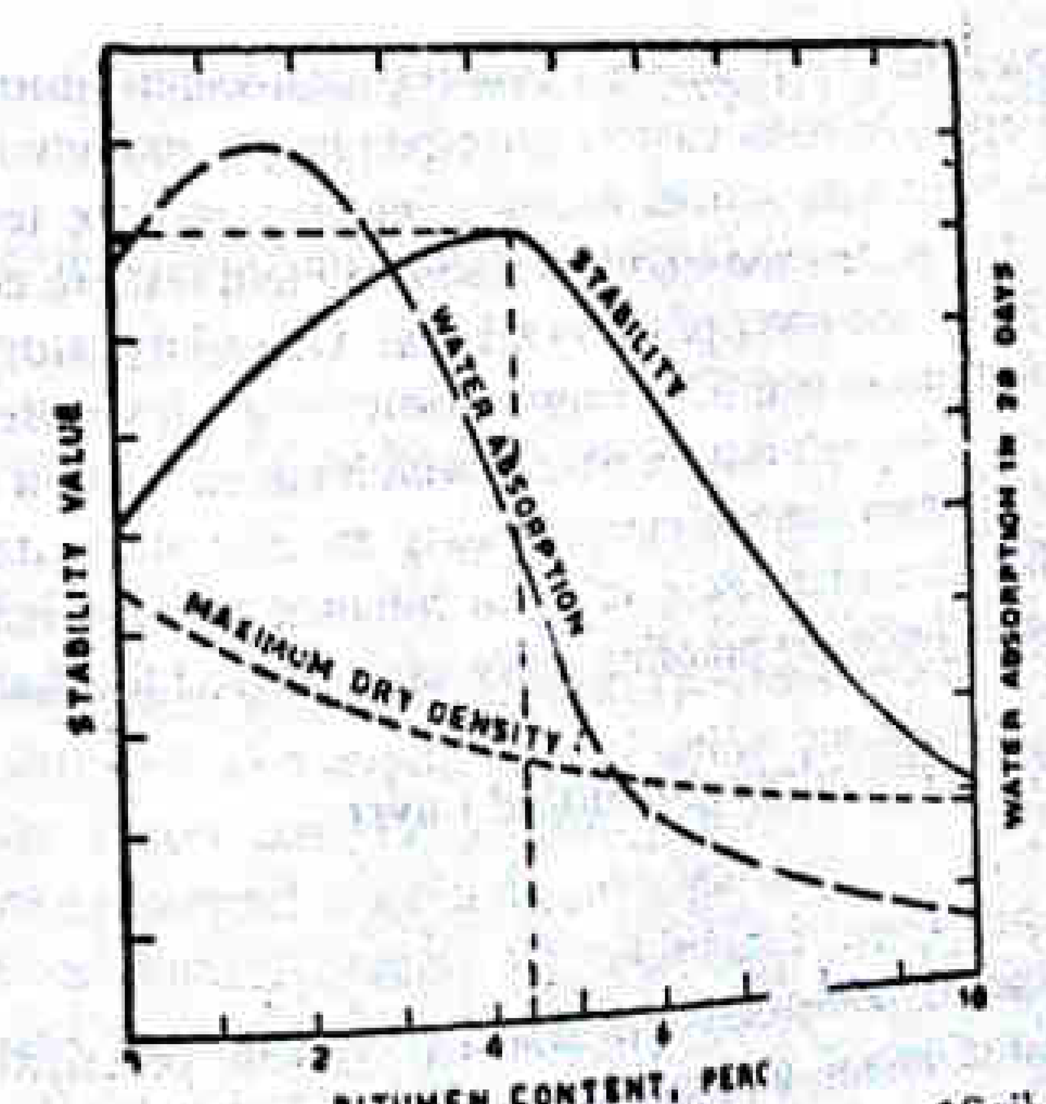


Fig. 9.7 Effect of Bitumen Concrete on Pr[illegible] of Soil [illegible]nen

content for maximum stability generally ranges from 4 to 6 percent by weight of dry soil, depending on the soil properties.

Mixing

Improved type of mixing with low mixing period may be preferred. In order to make mixing possible and to disperse bitumen in fine particles, it is necessary to first mix the soil with water before adding the cutback. Mixing temperatures (which is generally the atmospheric temperature itself) also affects the properties of the mix, depending on the type and grade of cutback and the soil type.

Compaction

Better the compaction, higher will be the stability and resistance to absorb water. The compaction characteristics and the properties of the resulting mix depend on the compacting moisture content and temperature, aeration of the mix before compaction and the amount and type of compaction. The optimum moisture content values corresponding to maximum dry density, maximum soaked stability and minimum water absorption for a soil bitumen mix may differ slightly depending on the proportions and properties of the mix constituents.

Curing

By curing soil bitumen, the water and the volatiles (of the solvent used in cutback) are allowed to evaporate thereby allowing the bitumen to be effective to impart the binding and water proofing actions. The curing period required to achieve desired stability and loss in weight would depend on curing temperature, relative humidity and soil type.

Additivies

Anti-stripping and reactive chemical additives have been tried to improve the properties of the mixes with varying degree of success. Portland cement is also some times used along with soil-bitumen to increase the stability of the mix.

9.5.3 Design of Soil-bitumen Mix

There is no standard of mix design. However soil-water-bitumen mix are generally compacted at the optimum water content corresponding to maximum dry density. These specimens are prepared with various bitumen contents and are tested for stability and water absorption. C.B.R. test and modified Hubbard Field test are considered suitable for evaluating the stability and water proofing effects. Other laboratory tests which may be used in soil-bitumen mixes free from course particles are Iowa Bearing Value test and cone penetration test. A graph may be plotted with bitumen content *versus* stability value and the optimum bitumen content corresponding to maximum stability value may be found. Either this value or a lower value of bitumen content may be adopted if the desired stability value and water-proofing action are achieved for that mix.

9.5.4 Construction of Bituminous Stabilized Layer

Materials

The selected soil is pulverised and stacked. From practical considerations the following properties of the soil are suggested for proper stabilization :

Passing 4.75 mm sieve	< 50 percent
Passing 0.425 mm sieve	35 – 100 percent
Passing 0.075 mm sieve	10 – 50 percent
Liquid limit	< 40 percent
Plasticity index	< 18 percent

Plants and Equipment

The plants and equipment needed are for carifying, pulverizing, mixing and compaction.

Construction Steps

(i) The soil to be stabilized is pulverized.

(ii) Water is added to soil and is mixed.

(iii) Cutback or emulsion is now added and the moist soil is re-mixed for proper distribution of bitumen. The quantity of water to be added in step (ii) would be lesser when emulsion is used for stabilization.

(iv) The mix is spread, graded and compacted.

(v) The compacted layer is allowed to cure when the moisture and volatiles of the solvent evaporate.

(vi) Field control tests include the following :

(a) checking of pulverisation of wet mixed soil

(b) checking of moisture content and bitumen content before compaction.

(c) checking of dry density after compaction.

9.6 SPECIAL PROBLEMS IN SOIL STABILIZATION WORK

9.6.1 General

The main problems in stabilized road construction projects may be divided into three aspects :

(i) the choice of the stabilization technique to be adopted.

(ii) the design of the stabilized mix.

(iii) the thickness design of the stabilized layer/layers.

Choice of the Stabilization Technique

The choice of the stabilization technique depends upon several factors such as (a) the type of soil to be stabilized, (b) availability of other construction materials and admixtures at the locality at low cost, (c) volume, type and distribution of traffic, (d) climate, drainage and environmental conditions, (e) requirements of the pavement layer, whether sub-base, base or surface, course (f) construction equipment available, (g) urgency of the project and (h) other limitations for the construction including working conditions. Apart from all these factors, the overall funds available for the project would also govern the final choice of the stabilization technique.

Mix Design

After the stabilization techniques to be adopted is broadly decided keeping in view all the factors mentioned above, the next requirement is the mix design. The mix design should take into consideration the stability and durability requirements of the pavement layer for which the design is to be made. Other factors to be considered in the mix design are the materials including the stabilizers available and the overall economy of the various mixes which fulfil the design requirements. Use of additives may also be considered to increase the stability or the rate of gain in strength of the mix. Scope of combination of stabilization methods, or *complex methods* may be investigated to compare the economy and other advantages with respect to the conventional methods of stabilization.

It is necessary to standardize the mix design procedure and design criteria of the various stabilized mixes. The mix property should be designated in terms of suitable strength value so that the same may be subsequently utilised in thickness design problems. For mixes with little flexural strength, the strength characteristics may conveniently be designated by compressive strength, C.B.R. or by any other suitable stability test. Stabilized mixes with noteworthy flexural strength such as soil-cement, pozzolanic concrete mix etc. should be tested for flexural strength also using beam specimens so as to enable the designer to use these strength values for designing the thickness base course slabs.

Thickness Design of Stabilized Layer

The aspect which is often given less significance in stabilization projects, the thickness design requirement of the stabilized layer. The thickness requirement of the stabilized layer is generally adopted either as equal to the granular base course thickness or by using some modification factor for gravel equivalency. However the thickness design of the stabilized layer in terms of the granular base course or the WBM layer depends on several factors such as the type and properties of the mix, the traffic, climatic and environmental conditions, and the design life of the pavement. The common concept of equal pressure criterion seems to be inadequate for deciding gravel equivalency factors of stabilized materials.

The stabilized mixes which are equivalent to conventional granular base course materials from structural stability and performance criteria, do not pose a problem for designing the thickness requirement of the pavement layer. But mixes which are inferior in performance or weaker structurally should be considered separately while designing the thickness of the layer. In the case of semi rigid mixes, the thickness requirement should be less than the granular base course. It is very difficult to assess the thickness requirements of these materials. So far there is no design method which is accepted as suitable for the design of such stabilized materials.

Soil-cement and pozzolanic concrete pavements may be designed as semi-rigid pavements, their deflection, pressure distribution and fatigue characteristics are considerably different from other flexible pavement materials and stabilized mixes. Thus careful consideration and extensive studies are needed before adopting equivalency factor for thickness design of stabilized mixes in terms of granular base course thickness.

In some soils the stabilizer requirements and the cost increases to such a great extent that stabilization may work out to be costlier than conventional construction, unless suitable steps are taken. Highly clayey soils like *Black Cotton Soils* are considered to be a problem-soil in India. Similarly stabilization of *desert sand* for road construction is also a typical problem. Hence these two problems of stabilization are briefly discussed in the subsequent paragraphs.

9.6.2 Stabilization of Black Cotton Soils

Black Cotton (BC) soils are highly clayey soils, grayish to blackish in colour found in several states in India. In several BC soil areas suitable road aggregates are to be transported from distant places, thus increasing the cost of conventional type of pavement layers. Typical behaviour of these soils under different climatic conditions has made the construction and maintenance of roads not only expensive, but also difficult. The pavements constructed in black cotton soil areas are found to suffer from early failures. In flexible pavements with heavy traffic excessive unevenness, ruts, waves and corrugations are formed almost after every monsoon season, resulting in heavy cost of maintenance demand every year.

The black cotton soils are found to contain *montmorillonite* clay mineral which has high expansive characteristics. The colloidal clay content in BC soils is upto 50 percent and the fraction passing 0.075 mm sieve ranges from 70 to 100 percent. The liquid limit and plasticity index values ranges from 40 to 100 percent and 20 to 60 respectively. BC soils have low shrinkage limit (10 to 15 percent) and high OMC (25 to 30 percent). All these properties render the soil to be highly sensitive to moisture changes, compressible and plastic in nature.

From stabilization point of view, the following are the main problems in the case of BC soils :

(i) It is very difficult to pulverize the soil as the dry lumps are different to brake due to high dry strength and the wet soil is too sticky and unmanageable.

(ii) There is excessive variation in volume and stability with variation in water content.

(iii) There is considerable shrinkage on drying, resulting in the formation of extensive cracks. B.C. soil compacted at OMC will also shrink when dried as shrinkage limit is much lower than OMC.

(iv) The BC soil exerts high swelling pressure from below on being soaked.

(v) Conventional construction materials like hard aggregates may not be available within easy reach.

The BC soils are very poor and undependable subgrade material. Hence the main problem is to treat the subgrade soil itself such that the undesirable characteristics are modified by a suitable stabilization technique. Also suitable methods of constructing sub-base and base courses are to be decided based on practical considerations.

Of the various methods of stabilization, the most effective method to stabilize BC soils are by using lime along with suitable additives; when necessary. Also thermal stabilization is quite effective; however heat treatment of the entire soil for stabilization seems to be difficult and expensive. The cement requirement for satisfactory stabilization of BC soil is so high (15 to 25 percent) that it is not advisable to use Portland cement, all by itself, for stabilization.

By the addition of suitable proportion of lime to the BC soil and allowing it to react for a day or two, the modification in the properties take place. The following are the predominant modification in lime treated BC soils :

(i) Plasticity index decreases to almost zero, and the soil behaves as non-plastic soil. Also the soil becomes friable in nature and it becomes easy to pulverize the lumps.

(ii) The affinity with water decreases and there are less changes in volume and stability with changes in moisture content.

(iii) The soaked stability value is increased.

(iv) The shrinkage limit is increased and there is less shrinkage after compaction.

(v) The cement requirement of the lime-treated soil is considerably lesser than the untreated BC soil.

All these modifications are desirable changes for stabilization of the soil for use as a subgrade, sub-base and base course materials. The following are the recommendations for satisfactory stabilization of BC soils for road construction :

(i) The soil subgrade may be treated with a small proportion of lime (of the order of about one percent by weight) up to the desired depth.

(ii) Sub-base may be constructed by a layer of well compacted soil-lime. Lime stabilization work may be carried out in two stages, first by pre-conditioning the clay using about 2 to 3 percent lime and then by repulverising the modified soil, adding remaining lime (2 to 3 percent) and required water and compacting to a high density. Thickness of the layer depends on design considerations and construction equipment.

(iii) Base course may be prepared by using soil-lime plus some additive. Along with the BC soil treated with 3 to 6 percent lime (in two stages), addition of 3 to 5 percent Portland cement would result in a strong base course layer. Instead of Portland cement, pozzolanic materials like flyash, surkhi, heat-treated BC soils or even cinder dust may be used. Soil-lime-flyash and soil-lime heat treated BC soil have shown great promise in making good and economical base course using BC Soils. The stabilized soil mix should be properly designed. Where good road aggregates are available locally, WBM bases may be constructed.

(iv) Bituminous surface course, preferably a superior type (premix type) should be used as a wearing course.

9.6.3 Stabilization of Desert Sand

There are large deposits of desert sand in regions of Rajasthan and other places in India. It is really a great problem to construct roads across the desert mainly because of non availability of other suitable construction material. There is also acute scarcity of water in the desert region. Hence a suitable stabilization technique seems to be the only economical solution.

The desert sand deposits consist of fine grained uniformly graded sand with, rounded particles. This renders the desert sand with poor stability. The cement requirement for satisfactory stabilization is also very high in such soil. Due to the scarcity of water, soil-cement-stabilization is all the more difficult as considerable water is needed for soil cement base course construction. Keeping all these problems in view, bituminous stabilization seems to be a suitable solution.

Use of hot sand-bitumen would result in satisfactory mix, provided some material including filler can be added to give a proper gradation of the mix. In this connection mixing of locally available kankar dust has been found to give satisfactory results. However use of hot sand-bitumen mix is not economical for sub-base and base course construction. If cutback is to be used, the requirement of mixing water content would be considerable.

The most promising bituminous material in desert region seems to be the emulsion. As the emulsion contains about 50 percent water, the additional quantity of water needed for mixing would be very less. During curing the water evaporates, the emulsion breaks down and the bitumen stabilizes the sand. The stability of the mix could be improves by the addition of kankar powder, and other material to improve the gradation. Also suitable additives to bituminous emulsion may be tried.

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PROBLEMS

1. Discuss the scope of soil stabilization in road construction.
2. Explain the mechanics of soil stabilization.
3. Explain the principle, application and method of construction of mechanical soil stabilization.
4. Discuss the properties of soil-aggregates mixtures and the factors affecting the properties of mechanical stabilized soil.
5. Discuss the scope of soft aggregate in soil stabilization. Explain Mehra's method of stabilization.
6. Explain the factors in design of mix for mechanical soil stabilization.
7. Discuss the principles and scope of soil-cement stabilization.
8. What are the factors influencing the properties of soil-cement ? Explain how soil-cement mix is designed.
9. How is soil-cement base course constructed ? Give details.
10. Explain the principle, scope and factors affecting the properties of soil-lime stabilization.
11. Enumerate the construction procedure of lime stabilized base course.

12. Discuss the principles and application of soil-bitumen. What are the factors affecting the properties of soil bitumen ?
13. Specify how soil-bitumen mix is designed and the stabilized road constructed.
14. Discuss the problems in stabilization of :

    (a) Black cotton soils

    (b) Desert sands

    Suggest suitable method of stabilization.

Chapter 10

Highway Maintenance

10.1 INTRODUCTION

10.1.1 Need for Highway Maintenance

Road maintenance is one of the important components of the entire road system. The maintenance operations involve the assessment of road condition, diagnosis of the problem and adopting the most appropriate maintenance steps. Even if the highways are well designed and constructed, they may require maintenance; the extent of which will depend on several factors including the pavement type. Various types of failures in pavements ranging from minor and localised failure to major and general failures do take place on roads. The failures may be due to one or a combination of several causes.

10.1.2 General Causes of Pavement Failures

Some of the general causes of pavement failures needing maintenance measures may be classified as given below :

(a) Defects in the quality of materials used.

(b) Defects in construction method and quality control during construction.

(c) Inadequate surface or subsurface drainage in the locality resulting in the stagnation of water in the subgrade or in any of the pavement layers.

(d) Increase in the magnitude of wheel loads and the number of load repetitions due to increase in traffic volume.

(e) Settlement of foundation of embankment of the fill material itself.

(f) Environmental factors including heavy rainfall, soil erosion, high water table, snow fall, frost action, etc.

10.1.3 Classification of Maintenance Works

The various items of highway maintenance works may be broadly classified under three heads :

(a) Routine maintenance/repairs : These include filling up of pot holes and patch

repairs, maintenance of shoulders and the cross slope, up-keep of the road side drains and clearing choked culverts, maintenance of miscellaneous items like road signs, arboriculture, inspection bungalows, etc.

(b) Periodic maintenance : These include renewals of wearing course of pavement surface and preventive maintenance of various items

(c) Special repairs : These include strengthening of pavement structure or overlay construction, reconstruction of pavement, widening of roads, repairs of damages caused by floods, providing additional safety measures like islands, sings etc.

10.1.4 Maintenance Management System

The type and extent of maintenance requirement for a road depend on the serviceability standard laid down, the maintenance needs funds available and the priorities for the maintenance operations. As several interlinked factors are involved in the maintenance works of road net work consisting of different categories of roads, a system approach is appropriate for the road maintenance management. The various factors to be included in the maintenance management system are :

(i) Minimum acceptable serviceability standards for the maintenance of different categories of roads.

(ii) Field surveys for the evaluation of maintenance requirements.

(iii) Various factors influencing the maintenance needs such as subgrade soil, drainage, climate, traffic, environmental condition, etc.

(iv) Estimation of rate of deterioration of the pavement under the prevailing set of conditions.

(v) Type and extent of maintenance requirements and various possible alternatives and their economic evaluation.

(vi) Availability of funds.

(vii) Maintenance cost, availability of materials, man power and equipment.

(viii) Need based allocation for optimum utilization of inputs and fixing maintenance priorities.

10.2 PAVEMENT FAILURES

10.2.1 General

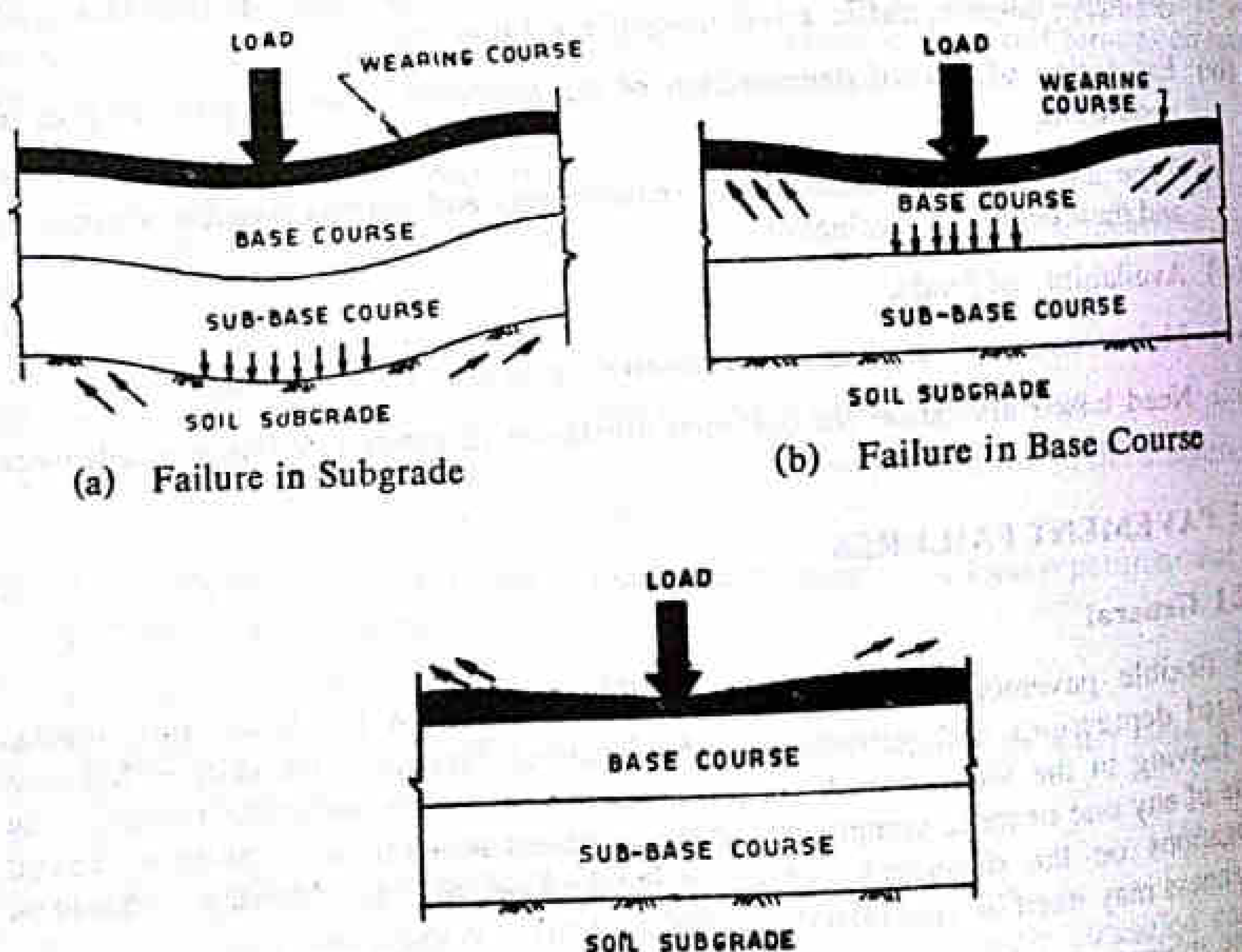
A flexible pavement failure is defined by formation of pot holes, ruts, cracks, localized depressions and settlements. The localized depression normally is followed with heaving in the vicinity. The sequence develops a wavy pavement surface. The failure of any one or more components of the pavement structure develops the waves and corrugations on the pavement surface or longitudinal ruts and shoving. Pavement unevenness may itself be considered, as a failure, when it is excessive.

The subject of pavement distress and failure is considered complex as several factors contribute to the pavement deterioration and failure. The aging and oxidation of bituminous films lead to the deterioration of bituminous pavements. Detrimental actions in pavements are rapidly increased when excess water is retained in the void spaces of bituminous pavements or in the cracks and joint of the cement concrete pavements.

The cement concrete pavements may develop cracks and deteriorate due to repeated loads and fatigue effects. A rigid pavement failure is observed by the development of structural crack of break resulting in progressive subsidence of some portions of pavement. Moderate irregularities in the supporting layers beneath the cement concrete pavements are sustained due to inherent bending strength of these pavements. Rigid pavements are therefore capable of withstanding slight variations in the underlying support and they bridge the localized gaps moderately. It is the combination of many factors that induce the failure conditions in the rigid pavement. Due to the temperature effects, the newly constructed cement concrete pavements may also crack even if no vehicle moves on them. Often failure of rigid pavements start from joints, corners and edges of slabs.

10.2.2 Failures in Flexible Pavements

As stated above, the localized settlement of any one component layer of the flexible pavement structure could be enough to cause pavement failure. This demands that each one of the layers should be carefully designed and laid. Thus to maintain the stability of the pavement structure as a whole, each layer should be stable within itself and thereby make the total pavement maintain its stability. Figure 10.1 illustrates the above concept. Figure 10.1-a, b and c illustrate the failures in the soil subgrade, base course and the surface or wearing course. It may be seen that ultimately there is surface deformation when failure takes place either in subgrade or base or surface.



(a) Failure in Subgrade

(b) Failure in Base Course

(c) Failure in Wearing Course

Arrows indicate the direction of up heaval due to the movement of material from the layer

Fig. 10.1 Failure in Flexible Pavement

Failures in Subgrade

One of the prime cause of flexible pavement failure is excessive deformation in subgrade soil. This can be noticed in the form of excessive undulations or waves and corrugations in the pavement surface and also depressions followed by heaving of pavement surface. The lateral shoving of pavement near the edge along the wheel path of vehicles is due to insufficient bearing capacity or a shear failure in subgrade soil. Excessive unevenness of pavement surface is considered as pavement failure.

The failure of subgrade may be attributed due to two basic reasons :

Inadequate stability

Excessive stress application

Inadequate stability may be due to inherent weakness of the soil itself or excessive moisture or improper compaction. Stability is the resistance to deformation under the stress. Excessive stress application is due to inadequate pavement thickness or loads in excess of design value. The deformation of soil subgrade and other pavement materials are found to increase with increase in number of load repetitions.

If the applied stress on the subgrade or pavement is very low when compared to its bearing capacity, the deformation due to the load would be *elastic* or fully recovered when the load is released. If the compaction of the layers is not adequate with reference to subsequent loading, part of the deformation may be permanent due to compaction of soil, this may be called *consolidation deformation.* But if the applied stress is excessive with respect to the stability and if plastic flow takes place as in the case of wet clayey soil, this deformation is called *plastic deformation* and is not even partly recoverable. These have been illustrated in Fig. 10.2.

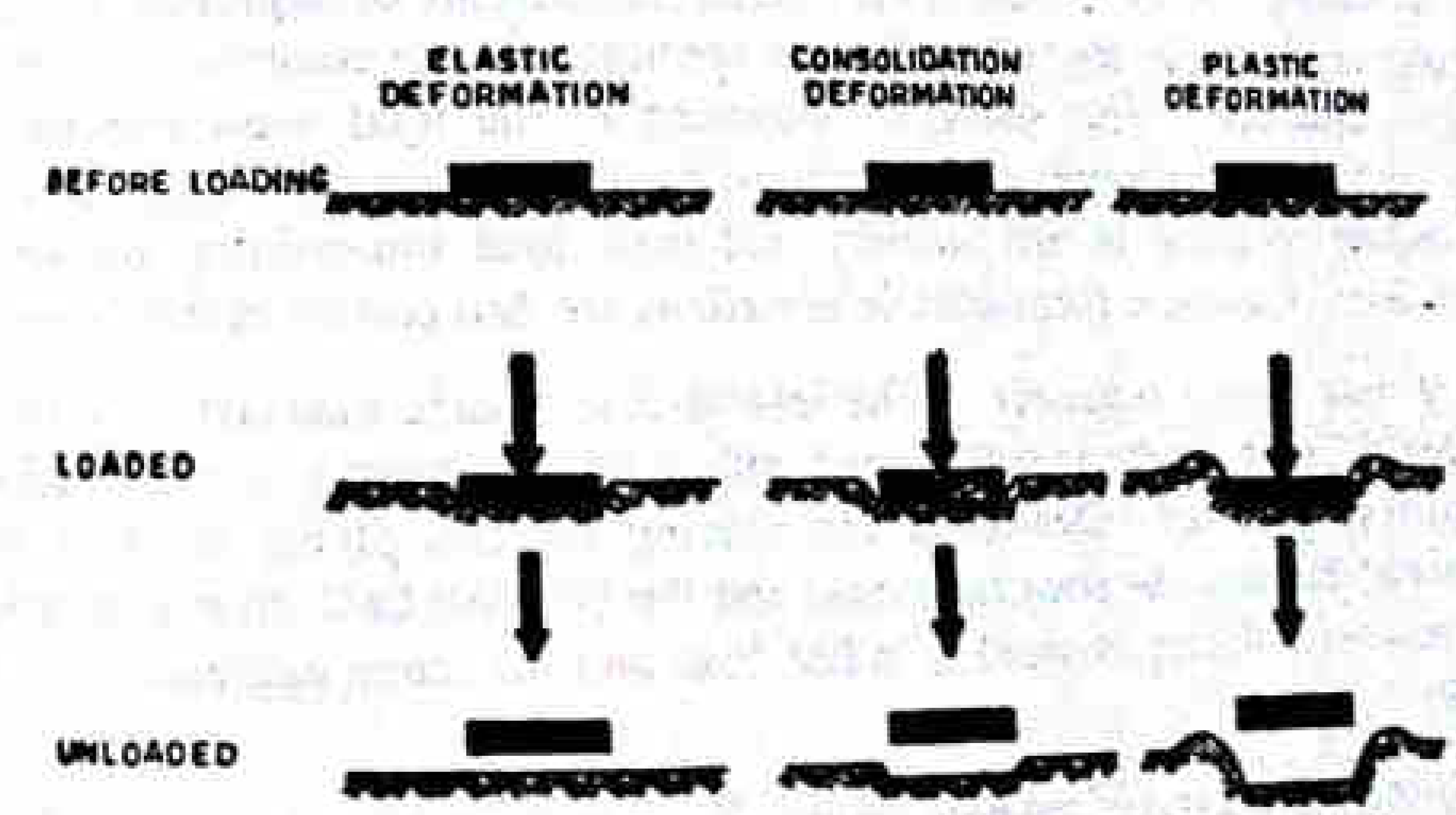


Fig. 10.2 Soil Deformation under Loads

The type of damage in flexible pavement that can be caused by traffic due to subgrade failure or due to inadequate and improper compaction of subgrade and other pavement layers has been illustrated in Fig. 10.3.

Failures in Sub-base or Base Courses

Following are the chief types of sub-base or base course failures :

(i) Inadequate stability or strength.

(ii) Loss of binding action.

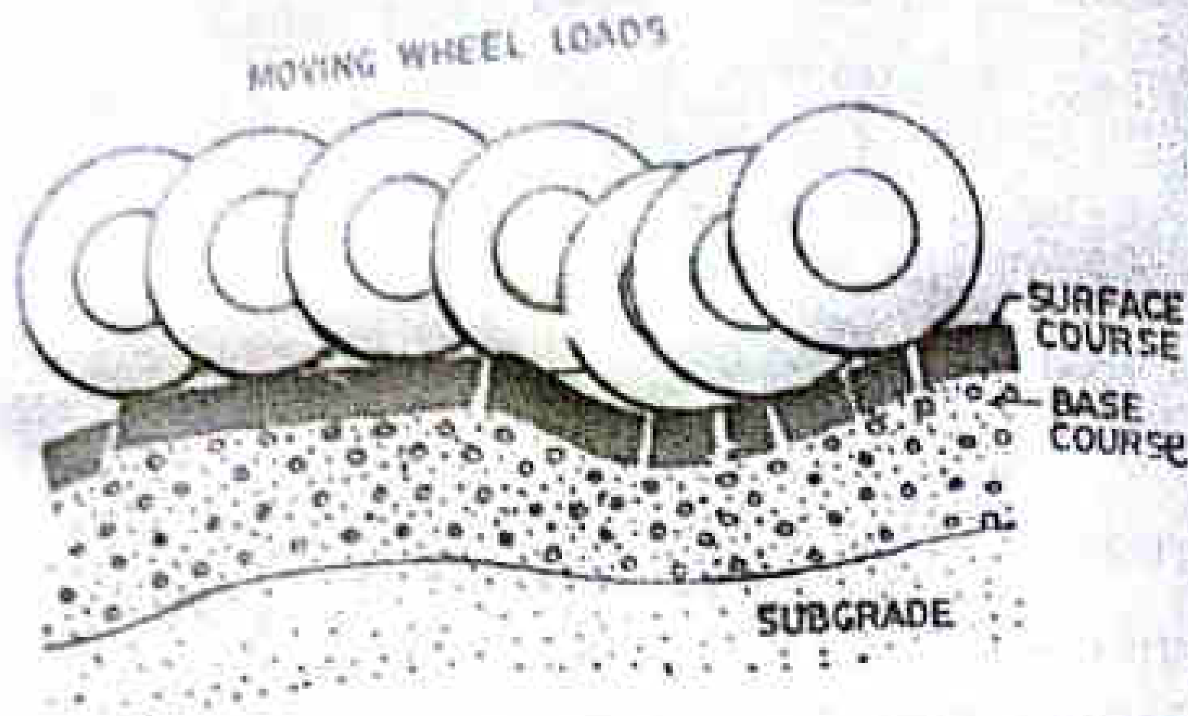


Fig. 10.3 Failure due to Improper Compaction of Subgrade and Pavement Layers

(iii) Loss of base course materials.

(iv) Inadequate wearing course.

(v) Use of inferior materials and crushing of base course materials.

(vi) Lack of lateral confinement for the granular base course.

Inadequate stability or strength : Poor mix proportioning or inadequate thickness are main reasons for the lack of stability or strength of sub-base or base course. Soft varieties of stone aggregates also make the base course layer weak. Improper quality control during the construction results in poor base course.

Loss of binding action : Due to the internal movements of aggregate in sub-base or base course layers under the repeated stress applications, the composite structure of the layers gets disturbed. This results in loosening of the total mass and formation of alligator cracks on the bituminous surfacing of flexible pavements. There is also loss of binding action resulting in low stability and poor load transmitting property of the pavement layer. Excessive permanent deformations are thus caused in this layer.

Loss of base course materials : The loss of base course materials is only possible when either the base course is not covered with a wearing course or the wearing course has completely worn out. Due to the fast moving vehicles plying on road, there is a suction caused between the pneumatic tyres and the exposed base course material. This causes removal of binding material in WBM base and the stone aggregates are left in a loose state.

The exposed aggregates of the base course also may form dust due to abrading action and attrition. With further use of such pavement sections, there is loss of stone aggregates forming *pot holes*. The removal of materials is called *ravelling*. Dust *nuisance* also develops under these circumstances. The preventive measures include repair of the pot holes and by painting the surface with dust pallivative or providing a suitable surface treatment. In WBM roads, it is conventional to spread a thin layer of soil to serve as the binding material after the rains, which also forms a suitable cushion to prevent the raveling of stones.

Inadequate wearing course : Absence of wearing course or inadequate thickness of stability of wearing course exposes the base course to the damaging effects of climatic variations mainly due to rains, frost action and the traffic.

Depending upon the type, intensity and volume of traffic, a suitable type and thickness of wearing course is provided over the base course. Bullock cart traffic causes damaging effects to the surface course. Pervious wearing course also permits the surface water to seep through and soften the base course thus weakening it.

Use of inferior materials : Many failures, mainly structural failures are attributed due to the use of inferior materials in the paving jobs. Some materials exhibit satisfactory characteristics initially, but show rapid deterioration due to weathering. The suitability of paving materials should be judged by the number of test listed in Chapter 6. *Highway Materials* and specifications for construction as listed in Chapter 8, *Highway Construction*.

Failure of wearing course : Failure of wearing course are observed due to lack of proper mix design. Improper gradation of aggregates, inadequate binder content and inferior type of binder result in a poor bituminous surfacing. Besides the design aspect, the bituminous construction requires a high degree of quality control, since over or under estimated binder content, are both greatly damaging to the resulting paving mix including temperature controls.

Volatilization and oxidation of binder also make the bituminous surfacing brittle and cause cracking of the pavement surface which further allows seepage of rain water to harm the underlying layers.

10.2.3 Typical Flexible Pavement Failures

Following are some of the typical flexible pavement failures :

(i) Alligator (map) cracking

(ii) Consolidation of pavement layers

(iii) Shear failure

(iv) Longitudinal cracking

(v) Frost heaving

(vi) Lack of binding (keying) to the lower course.

(vii) Reflection cracking

(viii) Formation of waves and corrugation.

Alligator (Map) Cracking

Figure 10.4 shows the general pattern of alligator or map cracking of the bituminous surfacing. This is the most common type of failure and occurs due to relative movement of pavement layer materials. This may be caused by the repeated application of heavy wheel loads resulting in fatigue failure or due to the moisture variations resulting in swelling and shrinkage of subgrade and other pavement materials. Localized weakness in the underlying base course would also cause a cracking of the surface course in this pattern.

Consolidation of pavement Layers

Formation of ruts are mainly attributed to the consolidation of one or more layers of pavement. The repeated application of loads along the same wheel path cause cumulative

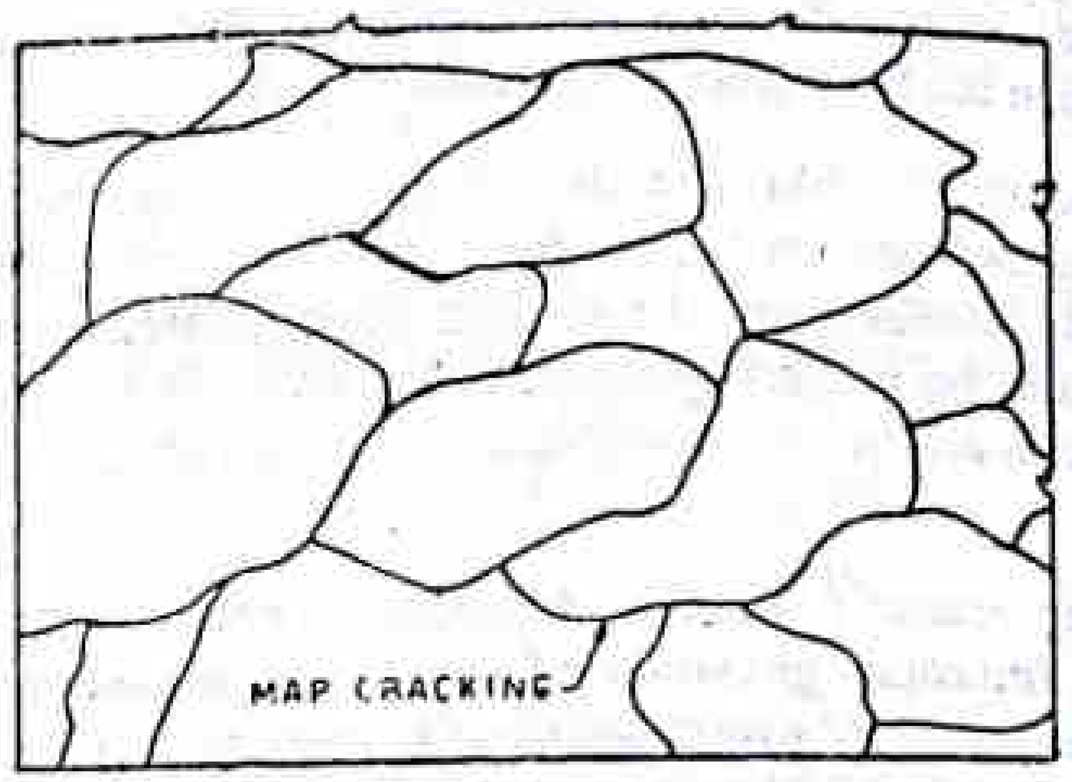


Fig. 10.4 Map Cracking

deformation resulting in *consolidation deformation* or longitudinal ruts. Shallow ruts on the surfacing course can also be due to wearing along the wheel path. Depending upon the depth and width of ruts, it can be estimated whether the consolidation deformation has been caused in the subgrade or in subsequent layers. A typical section of the pavement surface showing such failure is given in Fig. 10.5.

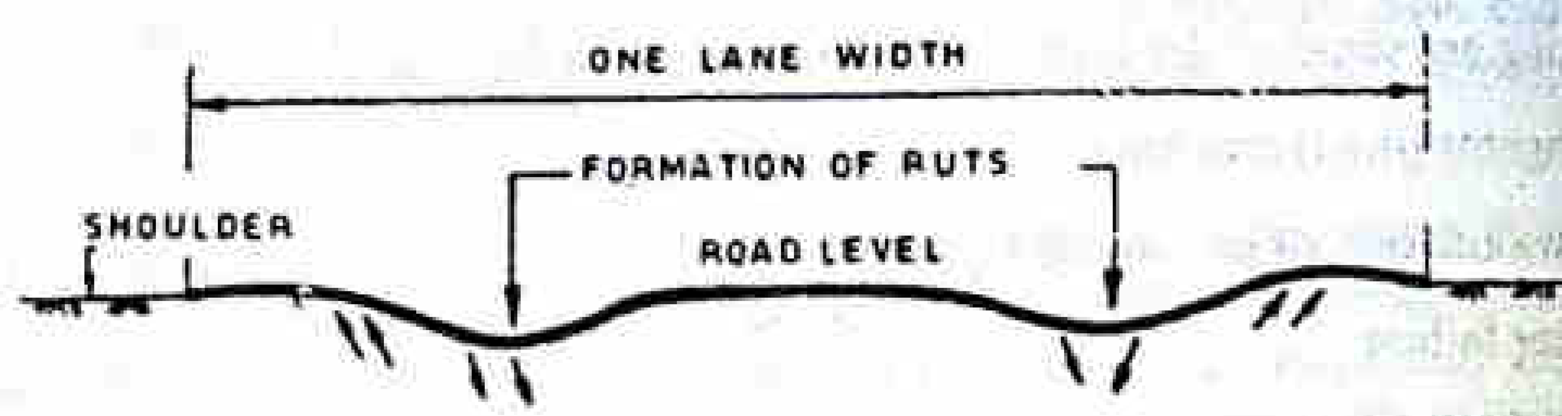


Fig. 10.5 Formation of Ruts

Shear Failure & Cracking

Shear failures are associated with the inherent weakness of the pavement mixtures, the shearing resistance being low due to inadequate stability or excessively heavy loading. The shear failure causes upheaval of pavement materials by forming a fracture or cracking. Figure 10.6 is a typical section showing this type of failure.

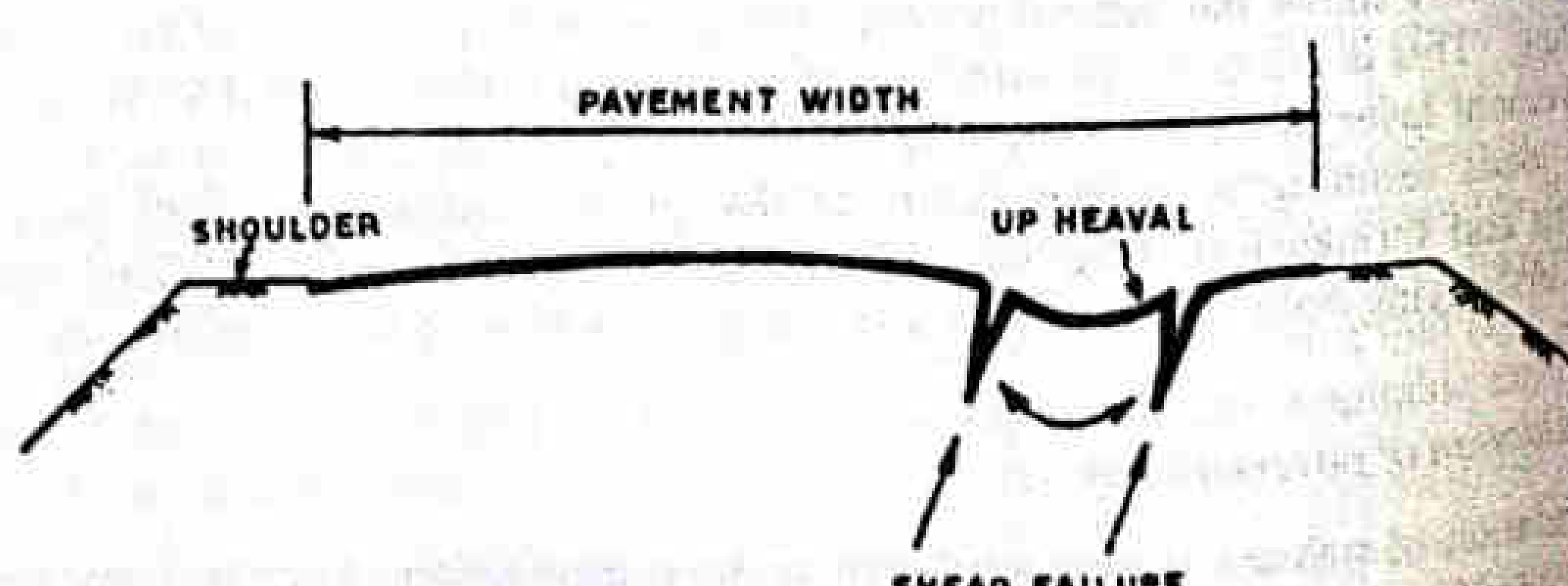


Fig. 10.6 Shear Failure Cracking

Longitudinal Cracking

Due to frost action and differential volume changes in subgrade longitudinal cracking is caused in pavement traversing through the full pavement thickness. Settlement of fill and sliding of side slopes also would cause this type of failure.

Frost Heaving

Frost heaving are often misunderstood for shear or other types of failures. In shear failure, the upheaval of portion of pavement is followed with a depression. In the case of frost heaving, there is mostly a localized heaving-up pavement portion depending upon the ground water and climatic conditions. See Fig. 10.7.

Fig. 10.7 Failure due to Frost Heave

Lack of Binding with Lower Layer

Slipping occurs when the surface course is not keyed/bound with the underlying base. This results in opening up and loss of pavement materials forming patches or pot holes. Such conditions are more frequent in case when the bituminous surfacing is provided over the existing cement concrete base course or soil-cement base course. This condition is more pronounced when the prime/tack coat in between two layers is lacking. The typical failure is shown in Fig. 10.8.

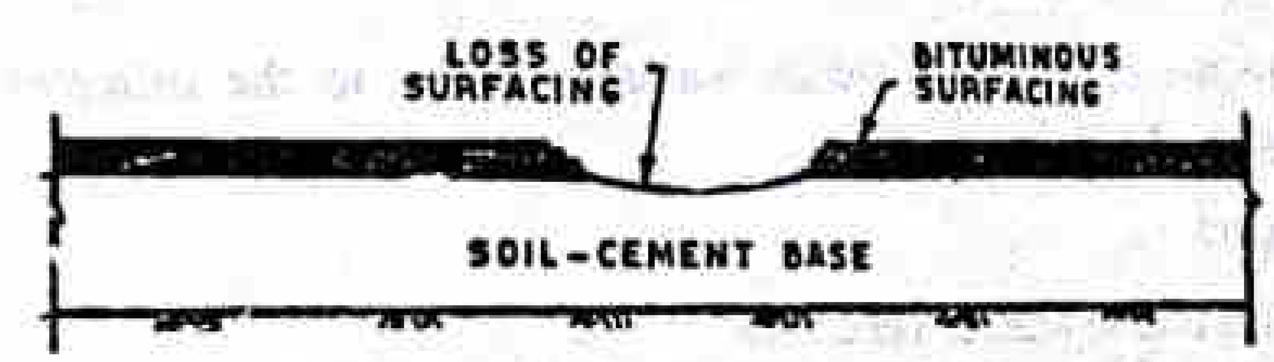


Fig. 10.8 Failure due to Lack of Binding

Reflection Cracking

This type of cracking is observed in bituminous overlays provided over existing cement concrete pavements. The crack pattern as existing in cement concrete pavements are mostly reflected on bituminous surfacing in the same pattern. Structural action of the total pavement section is not much influenced by the presence of reflection cracks but since the cracks appear at the surface, these allow surface water to seep through and cause damage to the soil subgrade or result in *mud pumping*. See Fig. 10.9.

10.2.4 Failure in Cement Concrete pavements

Failure of cement concrete pavements are recognized mainly by the formation of structural cracking. The failures are mainly due to two factors.

(i) Deficiency of pavement materials

(ii) Structural inadequacy of the pavement system.

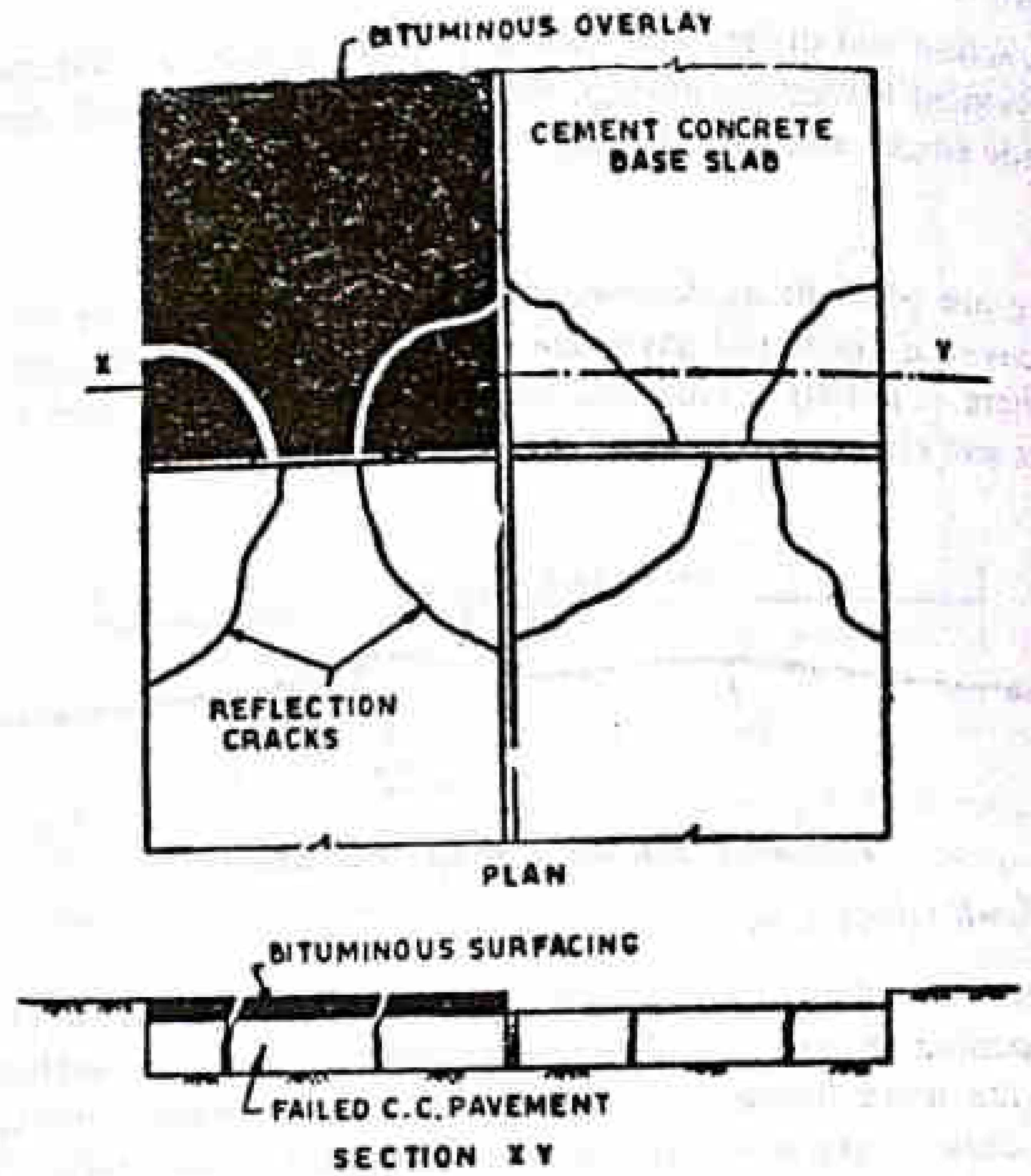


Fig. 10.9 Formation of Reflection Cracks

Deficiency of Pavement Materials

Following are the chief causes which would give rise to the different defects or failures of cement concrete pavement :

(i) Soft aggregates

(ii) Poor workmanship in joint construction

(iii) Poor joint filler and sealer material

(iv) Poor surface finish

(v) Improper and insufficient curing

The various defects that creep in due to the above are

(i) Disintegration of cement concrete

(ii) Formation of cracking

(iii) Spalling of joints

(iv) Poor riding surface

(v) Slippery surface

(vi) Formation of shrinkage cracks

(vii) Ingress of surface water and further progressive failures

Structural Inadequacy of Pavement System

Inadequate subgrade support pavement thickness would be a major cause of developing structural cracking in pavements. Following are the causes and types of failure which develop :

(i) Inadequate pavement thickness

(ii) Inadequate subgrade support and poor subgrade soil

(iii) Incorrect spacings of joints

Above would give rise to the failures of the following types :

(i) Cracking of slab corners

(ii) Cracking of pavements longitudinally.

(iii) Settlement of slabs

(iv) Widening of joints

(v) Mud pumping

10.2.5 Typical Rigid Pavement Failures

Following are some typical and basic types of failures in rigid pavements which are dealt here in detail :

(i) Scaling of cement concrete

(ii) Shrinkage cracks

(iii) Spalling of joints

(iv) Warping cracks

(v) Mud pumping

(vi) Structural cracks

Scaling of Cement Concrete

Scaling is observed in cement concrete pavement showing overall deterioration of the concrete. The scaling is mainly attributed due to the deficiency in the mix or presence of some chemical impurities which damage the mix. Further due to excessive vibration given to mix, the cement mortar comes to the top during construction and thus with use, the cement mortar gets abraded exposing the aggregate of the mix. This makes the pavement surface rough and shabby in appearance.

Shrinkage Cracks

During the curing operation of cement concrete pavements immediately after the construction, the shrinkage cracks normally develop. The placement of cracks are in longitudinal as well as in transverse direction.

Spalling of Joint

Sometimes when pre-formed filler materials are placed during casting of pavement slabs, the placement is some how dislocated and filler is thus placed at an angle. The concreting is completed without noticing this faulty alignment of the filler material. Thus this forms an overhang of a concrete layer on the top side and the joint later on shows excessive cracking and subsidence.

Warping Cracks

If the joints are not well designed to accommodate the warping of slabs at edges, this results in development of excessive stresses due to warping and the slab develops cracking at the edges in an irregular pattern. Hinge joints are generally provided for relieving the slabs of warping stresses. There is no structural defect due to the warping cracks if proper reinforcement is provided at the longitudinal and transverse joints as it takes care of the structural adequacy.

Mud Pumping

Mud pumping is recognised when the soil slurry ejects out through the joints and cracks of cement concrete pavement caused during the downward movement of slab under the heavy wheel loads. Following are the factors which cause the mud pumping :

(i) Extent of slab deflection

(ii) Type of subgrade soil

(iii) Amount of free water

Pumping is noticed just after the rains in cement concrete pavements that are placed on clayey soil subgrade. Due to the applications of repeated loads, initial spaces are developed underneath the pavement slabs and water infiltrates into these spaces through joints, cracks and edges of the pavements as shown in Fig. 10.10a. Since the soil is also of fine grained type, it holds water and forms the soil slurry or soil suspension in water or the *mud*.

Subsequent application of heavy wheel loads causes the pavement slab to deflect at critical locations and also forces out part of the mud each time, through the spaces in pavement joints, cracks or edge. See Fig. 10.10b. When more and more mud is ejected out, there is a substantial loss in fine grained soil from subgrade, resulting in considerable loss of subgrade support at these locations. With continued traffic movements, there is progressive increase in the wheel load stress in the pavement slab due to reduction in subgrade support, consequently cracks are developed and the pavement ultimately fails as illustrated in Fig. 10.10c. The pavement cracking due to mud pumping is generally a progressive type of failure in rigid pavements.

Inadequate pavement thickness for the amount and type of vehicles is the prime reason for the structural cracking. Largely, the pavements are found to crack at the corners and edges as shown in Fig. 10.9. Longitudinal and transverse cracks are also found to exist. It becomes quite difficult to differentiate the type of cracks. Generally, if it could be decided that the crack in the vicinity of joints or corners are not due to spalling or mud pumping, then the cracks are attributed to the structural inadequacy. The cracking in the interior regions are mainly due to the temperature stresses.

10.3 MAINTENANCE OF HIGHWAYS

10.3.1 General

After knowing the various defects that may cause pavement failures, it is necessary to study the different measures that one adopted to maintain and upkeep the pavement for their excellent functions. As a basic principle, it may be observed that a highway which is designed based on scientific standing would resist the various detrimental force and would need the minimum maintenance. Various maintenance operations are :

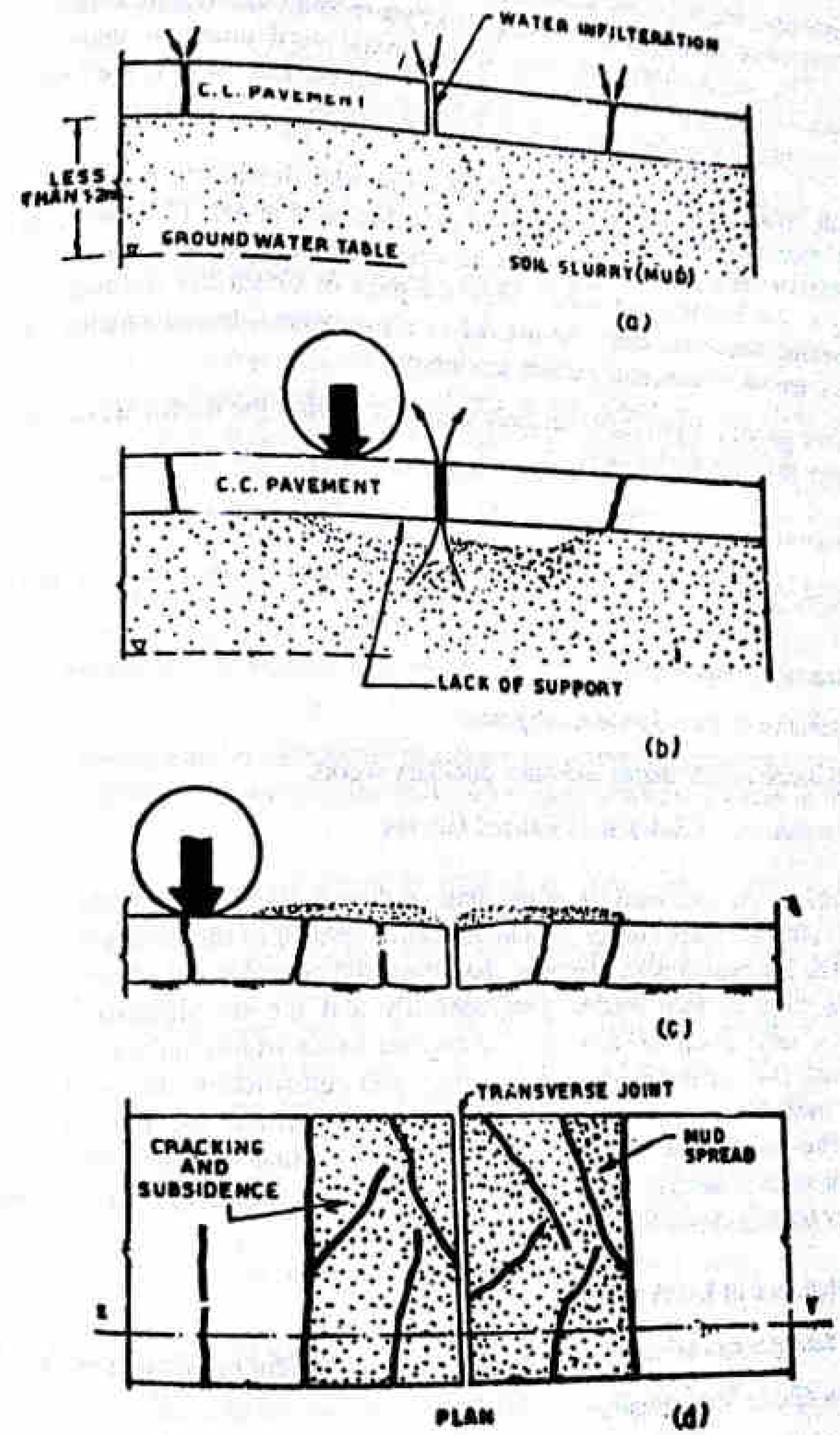


Fig. 10.10 Mud Pumping

(i) Routine maintenance

(ii) Periodic maintenance

(iii) Special repairs

Routine and periodic maintenances are needed for any type of road whether it is designed and constructed with scientific bias or not. Since the highways are exposed to the moving traffic and adverse climatic conditions, they would positively wear out. Even if the highways are not used, they also would need maintenance for their upkeep, as the factors that cause damage to the pavements are more than one. Traffic is one of the many factors. The longitudinal and cross drains would need attention under the routine maintenance work. One of the common items of maintenance work is the removal of silt, rubbish and weeds from the longitudinal and cross drains.

Special repairs and strengthening of pavement with overlays are needed to prevent pavement failures. Many defects that have been discussed in Art. 10.2 need special care and the maintenance jobs in such cases become even more complex than planning and designing a new highway net work. Special repairs of subsurface drainage system and cross drainage structures may be needed in some cases. Improvements of highway geometrics may also be included under special repairs.

When the pavement surface is severely damaged or after the useful service life, it may be necessary to renew the surface.

10.3.2 Routine Maintenance

Following is the brief list of various routine maintenance that may be required for highways :

(i) Upkeep of carriageway

(ii) Maintenance of shoulders and subgrade.

(iii) Maintenance of side drains and other ancillary works.

(iv) Patch repairs of pot holes and localised failures.

As a principle, the road network of the State in view is divided in number of sections and these sections are placed under the administrative control of the different Engineering divisions. The responsibility of planning, designing and constructing of new networks is obviously assigned to each section geographically and the maintenance also thus is assigned to the same groups or sections. The engineers thus would maintain the complete records of each case commencing from its design and construction stages. Unless such records are available at hand, it would be difficult to assess the real cause of the problems. The maintenance schedule listed above of routine type and the repairs are carried out at regular interval like (i) day to day and (ii) seasonal. The maintenance techniques are now discussed for different types of highways.

10.3.3 Maintenance of Earth Roads

The usual damages caused in the earth roads needing frequent maintenance are :

(i) formation of dust in dry weather

(ii) formation of longitudinal ruts along wheel path or vehicles

(iii) formation of cross ruts along the surface after monsoons due to surface water.

Thus, dust nuissance may be remedied by the following methods :

Frequent sprinkling of water

Treatment with calcium chloride

Use of other dust palliatives

Application of calcium chloride retain some water due to the hygroscopic nature of mix. Oiled earth roads are also common these days.

Periodical maintenance by spreading moist soil along ruts and reshaping of the camber is necessary. Formation of cross ruts may be due to excessive cross slope. However in areas of heavy rain fall, it may not be possible to avoid these in untreated earth road. Hence either these ruts should be repaired from time to time during and after the monsoon or a surface treatment or stabilized layer be provided on the top.

10.3.4 Maintenance of Water Bound Macadam Roads

WBM roads are damaged rapidly due to the heavy mixed traffic and adverse climatic conditions. In dry weather dust is formed and during rains mud is formed. The steel tyred bullock carts cause severe wear and tear to the WBM surface. The fast moving automobiles raise dust in dry weather and churn-up mud in wet conditions. Due to the combined effects of traffic and the rain water washing away the soil binder from the surface, the stone aggregates protrude out or get loose on the surface layer. Pot holes and ruts are also formed in localised spots of WBM roads.

To prevent the aggregate from getting loosened from the surface course, it is necessary to replace the soil binder periodically. This is done by spreading a thin layer of moist soil binder on the surface as a part of periodic maintenance, particularly after the monsoons.

Dust nuissance can be effectively prevented by providing bituminous surface dressing course over WBM pavement. However temporary measures include spraying of dust palliatives.

Pot holes and ruts formed should be patched up. The *patch repair work* is carried out by first cutting out a rectangular shape of the defective area to remove the stone up to the affected depth. Then with coarse aggregate of the same size (using part of the old aggregate and remaining fresh aggregate) the patch is filled up and compacted well by ramming such that the patched area is about 1 cm above the general pavement surface. This allows for the further compaction of this patched portion, under traffic. Wet soil binder is applied on the surface of the patched area to fill up the interstices and the surface is rammed again.

After the period of useful life of the WBM road or when the layer fails extensively, a resurfacing is called for. If the thickness requirement of the old road was inadequate, the thickness of the WBM pavement is increased during the resurfacing.

10.3.5 Maintenance of Bituminous Surfaces

Mainly the maintenance works of bituminous surfacing consists of :

(i) Patch repairs

(ii) Surface treatments

(iii) Resurfacing

Patch Repairs

Patch repairs are carried out on the damaged or improper road surface. Localized depression and *pot holes* may be formed in the surface layers due to defects in materials and construction.

Inadequate or defective binding materials causes removal of aggregates during monsoons. Patching may be done on affected localised areas or sections using a cold premix.

Pot Holes and Repairs

Pot holes are cut to rectangular shape and the affected material in the section is removed until the sound materials are encountered. The excavated patches are cleaned and painted with bituminous binder. A premixed material is then placed in the sections. Generally, cutback or emulsion is used as binder. Bituminous emulsions could be used even when the pavement surface and the aggregates are wet during monsoons. The material so placed in the pot hole, is well compacted by ramming to avoid any *ravelling*. The materials in pot holes are placed in layers of thickness of 6 cm or so. It is however necessary to replace the base course materials with similar new material if the failure has been detected in the base course layer also. The finished level of the patches is kept slightly above original level to allow for subsequent compaction under traffic.

Surface Treatment

Excess of bitumen in the surface materials *bleeds* and the pavement becomes patchy and slippery. Corrugations or rutting or shoving develop in such pavement surfaces. It is customary to spread *blotting* materials such as aggregate chips of maximum size of about 10 mm or coarse sand during summer. Necessary rolling is done to develop permanent bond between the existing surface and the new materials, after heating the surface if necessary.

The binders in the black top surfaces also get oxidized due to ageing. This develops minute cracking in the pavement surface. Bituminous wearing surface may also get worn out showing up mossaic of aggregates or rough surface to traffic and/or heavy rain fall. Such pavement surface are applied with a renewal coat such as surface dressing or seal coat. If the surface has been seriously dressing or seal coat. If the surface has been seriously damaged due to oxidization or volatilization of binder materials, it may be necessary to apply more than one layer of surface treatments.

Resurfacing

In the event when the pavement surface is totally worn out and develops a poor riding surface, it may be more economical to provide an additional surface coarse on the existing surface. In case the pavement is of inadequate thickness due to increase in traffic loads and strengthening is necessary, then an overlay of adequate thickness should be designed and constructed.

10.3.6 Special Repairs in Flexible Pavements

Waves and corrugations

Following are the factors which contribute to formation of waves and corrugations.

(a) *Defective rolling* : If the rolling during construction stage is improper thus leaving the formation of waves then the process being progressive, the wave formation would continue indefinitely. However the subsequent traffic operations would also cause similar effects if the rolling is inadequate during the construction stage.

(b) *Poor subgrade conditions* : Subgrade consisting of poor soils including highly plastic or organic soils and high water table close to subgrade surface may cause non-

uniform and inadequate subgrade stability. When boulders are used as soling course in such subgrades there is differential settlement or sinking of these stones. All these would contribute to formation of corrugated pavement surface.

(c) *Poor gradation or mix* : Defective gradation or mix for the surface layer is another factor which gives rise to the wave formation pushing and pulling caused due to the vehicular movement enhance the defect further more.

(d) *Compaction temperature* : Viscous state of the bitumen binder greatly depends upon the temperature and thus very high temperature during mixing and compaction (rolling) of bituminous mix would make the resulting pavement surface layers with low stability and wavy surface is formed during rolling.

(e) *Unstable underlying layers* : Weak underlying layers also cause the formation of waves due to repeated plying of vehicles on such road. Failure of any one of the pavement layers can cause surface deformations as illustrated in Fig. 10.1.

Remedial Measures

There appears to be no way to improve the road surface once the waves and corrugations are already formed. Usually another layer of surface course is laid after laying a levelling course. But often the waves and corrugation again develop, unless the basic reason for this problem is investigated and proper measure is taken.

If the instability of underlying layer is due to excessive moisture conditions, suitable subsurface drainage system is warranted to remedy the defect permanently. If the failure is due to improper compaction of the lower layers, this would need complete reconstruction. If the failure is due to subgrade soil which may be a highly plastic expansive clay, the solution may be by subgrade treatment using a modifying agent for stabilization as discussed in Art. 9.6.2.

Depending upon the failure of a given layer, the pavements would be reconstructed right from the defective stage. Since the above factors would be carefully observed in replacement of layers, the pavement section so constructed may offer satisfactory riding quality.

Skidding of pavement surfaces

Skid resistance property of pavement surface is essential requirement for highway safety. The skid resistance or the friction of the pavement surface may be measured using any one of the devices such as the pendulum type friction recorded or the skid testing device attached to test vehicle or the instrument mounted dynamic skid resistance tester towed by another vehicle. Skidding has been contributing to the large number of accidents on highways. Statistics show that about 40 percent of the accidents are due to the poor skid resistance offered by the pavement surface. It may not be possible to construct highways which would completely eliminate skidding of the vehicles. Investigation carried out on this aspect indicate that the sufficient degree of skid resistance can be built into the road surface and can be maintained through proper construction and maintenance controls.

Water, clay, dust, dry sand, oil and grease on the pavements are few factors which cause skidding. These materials on the pavement surface cause a reduction in grid between tyre and the pavement surface. Skidding is of three types. *(a) Straight skidding (b) impending skidding (c) sideway skidding*. The straight skidding occurs in the direction of travel when the sudden brakes are applied. Impending skidding is encountered when the braking is gradual and wheel continues to revolve. Sideway skidding occurs on curves where sufficient superelevation is not provided or when the coefficient of friction is inadequate.

Highways can develop sufficient skid resistance if they are maintained clean and dry. But the presence of water film, debris and polishing characteristics of aggregate influence the skid resistance properly. Rough surfaces or textures like those of gravel road, WBM and cement concrete roads offer sufficient amount of skid resistance. Bituminous pavements are more prone to skidding. Wheels do the polishing of the aggregates and excessive bitumen bleeding to the surface produces a smooth skid-prone surface when wet. The remedial measure mainly is renewal of the wearing surface.

In the case of bituminous surfacing with excessive bleeding, some times stone chips are spread after heating the surface and rolled to form a non-skid riding surface.

10.3.7 Maintenance of Cement Concrete Roads

It may be stated here that very little maintenance such as maintenance of joints only is needed for cement concrete roads, if they are well designed and constructed. Main defect in this type of road is formation of cracks. It is therefore necessary to examine the cracks and causes are ascertained before any remedial measure is adopted. Various types of cracking have been explained in Art. 10.2.2.

Treatment of Cracks

The cracks developed in cement concrete (CC) may be classified into two groups :

(i) *Temperature cracks*, which are initially fine cracks or hair-cracks formed across the slab, in between a pair of transverse or longitudinal joints, dividing the slab length into two or more approximately equal parts due to the temperature stresses like the shrinkage stress, warping stress, etc. in the slab.

(ii) *Structural cracks* formed near the edge and corner regions of the slabs, due to combined wheel load and warping stresses in the slab.

The presence of fine cracks only as such are not harmful and do not call for immediate maintenance. As the cracks due to the shrinkage in the CC pavement start from the bottom of the slab, by the time fine cracks are visible on the top of the slab, the cracks at the bottom portion would have got widened. Due to repeated application of heavy wheel loads and the variations in temperature and moisture conditions, the cracks get widened and further deterioration becomes rapid. Once the surface water starts getting into the pavement and the subgrade through the widened cracks, progressive failure of the pavement is imminent. Therefore before these cracks get wide enough to permit infiltration of water, they should be sealed off to prevent rapid deteriorations.

The dirt, sand and other loose particles at the cracks are thoroughly cleaned using a sharp tool, stiff brush and pressure blower. Kerosene oil is applied on the cleaned cracks to facilitate proper bonding of the sealing material. The cracks are then filled by suitable grade bituminous sealing compound, heated to liquid consistency. The sealer is placed upto about 3 mm above the level of the slab along the cracks and a layer of sand is spread over it to protect the sealer temporarily.

The formation of structural cracks in CC slabs should be viewed seriously and needs immediate attention, as these indicate possible beginning of pavement failure. First the cause of the failure should be investigated. If the failure is confined to one or a few slab

only at a particular location, and in general there are no structural cracks in other slabs, the failure may be localized one due to some weak spot in the subgrade or due to localized settlement of embankment or underground drainage problem. The maintenance work in such a case involves first remedy of the basic cause of the failure and then re-casting the failed slabs. In the case of general pavement distress indicating the start of structural failure of the pavement, immediate steps are to be taken to strengthen the CC pavement by a flexible or rigid overlay expeditiously before the structural cracks develop in other slabs also. It is not worth while to provide an overlay over a badly cracked or failed CC pavement as the riding surface becomes very unsatisfactory due to uneven settlement of the cracked and broken slabs. In such a case the only permanent solution is removal of the broken-up CC pavement slabs and re-construction of new flexible or rigid pavement.

Maintenance of Joints

Joints are the weakest parts in CC pavements. The efficiency of the pavement is determined by the proper functioning of the joints. Majority of the failure in the CC pavements are observes at or near the joints. Therefore, utmost care is to be taken to see that the filler and sealer materials are intact at the joints. During summer the joint sealer material is squeezed out of the expansion joints due to the expansion of the slabs, subsequently as the slabs contract during winter, the joint gap opens out and cracks are formed in the old sealer material. (See Fig. 8.2.6). Therefore, periodic maintenance of the joint sealer is essential both at expansion and contraction joints as a part of routine maintenance work of the CC pavement. The opened-up joints are cleaned with brush and refilled with suitable joint sealer material before the start of the rains.

The joint filler material at the expansion joints may get damaged or deteriorated after several years of pavement life. The repair consist of removal of the sealer and deteriorated filler and sealer materials from the expansion joints cleaning up, replacement with new filler board (provided with suitable grooves cut on the bottom half at the positions of the dowel bars) and sealing the top of the joints with suitable sealer material. It will be convenient to insert the new filler board at the expansion joints during winter season when the joint opening is widest.

10.3.8 Special Repairs of Cement Concrete Pavements

Mud Jacking or Lifting of Slabs

Once pavement starts pumping, the remedy for correcting it lies in providing the effective drainage. If the subsidence is localized then the same is repaired by patching the portions of slabs with bituminous mixes. Advanced countries adopt the procedures of *mud jacking*. The process consists of drilling number of holes 4 cm to 5 cm diameter 1.5 metre to 3 metre apart in the cement concrete slab. Grouting in such slabs is done under pressure through these holes. The grout normally used is either 1 : 3½ cement-sand mix or bitumen. For cement-sand mix, colloidal mix with sufficient water is prepared. The mix is thus injected through a pressure holes using the compressor. The slabs are thus raised from below by the pressure grout, upto the desired level.

10.4 PAVEMENT EVALUATION

10.4.1 General

Pavement evaluation involves a thorough study of various factors such as subgrade support, pavement composition and its thickness, traffic loading and environmental conditions. The primary objective of pavement condition evaluation is to assess as to whether and to what extent the pavement fulfils the intended requirements so that the maintenance and strengthening jobs could be planned in time. The studies therefore investigate the structural adequacy of pavements and also the requirements for providing safe and comfortable traffic operations.

There are various approaches and methods of pavement evaluations. The various methods may be broadly classified into two groups :

(i) Structural evaluation of pavements

(ii) Evaluation of pavement surface condition

10.4.2 Structural Evaluation of Pavements

The structural evaluation of both flexible and rigid pavement may be carried out by plate bearing test. The structural capacity of the pavement may be assessed by the load carried at a specified deflection of the plate or by the amount of deflection at a specified load on the plate.

Field investigations and tests carried out in various countries have shown that the performance of a flexible pavement is closely related to be elastic deflection under loads or its rebound deflection. Measurement of transient deflection of pavement under design wheel loads serves as an index of the pavement to carry traffic loads under the prevailing conditions. Of the various equipment used for the purpose, *Benkelman Beam* is most commonly used, as the measurements are simple and easy. This method was originally devised by *A. C. Benkelman* in U.S.A. Benkelman Beam measurements are preceded by a rating survey of the road so as to divide it into homogenous section of approximately similar serviceability. Assessment of flexible pavement overlay thickness requirement by Benkelman Beam method is explained in Art. 10.5.3. There are number of other *non-destructive* testing techniques for assessing the load carrying capacity of the pavements.

10.4.3 Evaluation of Pavement Surface Condition

The surface condition of flexible pavement may be evaluated by the unevenness, ruts, patches and cracks. The surface condition of rigid pavements may be assessed by the cracks developed and by faulty joints affecting the riding quality of the pavement.

The pavement unevenness may be measured using unevenness indicator, profilograph, profilometer or roughometer. An equipment capable of integrating the unevenness of pavement surface to a cumulative scale and that gives the Unevenness Index of the surface in cm/km length of road may be called, Bump Integrator or unevenness integrator. The U. S. Bureau of Public Roads Roughometer is one such device which could be towed by an automobile. In AASHO* road test, profilometer was used to record the variable slope angle of the surface formed by two probe wheels spaced 13.5 cm apart. The pavement unevenness criteria to indicate the pavement riding qualities expressed in terms of unevenness index recommended by Hollaway is given in Table 10.1.

The *pavement serviceability* concept was introduced at the AASHO Road Test for comparing relative performance of various test sections during periods. The present serviceability of a pavement is related to a pre-determined scale by a panel of judges sensitive to the wishes of motor vehicle users by actually riding over the pavement. The present serviceability rating (PSR) is the mean opinion of the members of the rating panel

*now AASHTO

Table 10.1

Unevenness index, cm/km	Riding quality
In old pavements	
Below 95	Excellent
95 to 119	Good
120 to 144	Fair
145 to 240	Poor (possible resurfacing)
Above 240	Very poor (resurfacing required)
In new pavements	
Below 120	Good (acceptable)
120 to 145	Fair (acceptable)
Above 145	Poor (not acceptable)

and this is correlated with the physical measurements such as longitudinal and transverse profile of the pavement, degree of cracking and patching etc. affecting pavement serviceability. Mathematical models are evolved for determining serviceability indices for predicting the road users serviceability rating of pavements based on the physical measurements made on the pavement surface. Two equations were formulated to give Present Serviceability Index (PSI) values for flexible and rigid pavements.

10.5 STRENGTHENING OF EXISTING PAVEMENTS

10.5.1 Objects

For the successful maintenance of pavements it is essential that they have adequate stability to withstand the design traffic under prevailing climatic and subgrade conditions. If the pavements have to support increased wheel loads and load repetitions, the pavements rapidly undergo the distress and no amount of routine and periodic maintenance can help them. Due to unexpected economic developments in the given region, the loading conditions may becomes severe and the alternative would be either to divert the traffic on some adjacent routes or to strengthen the existing pavements. Strengthening may be done by providing additional thickness of the pavement of adequate thickness in one or more layers over the existing pavement, which is called *overlay*. If the existing pavements have completely deteriorated, an overlay would not serve the purpose and the solution would be to remove the existing damaged pavement structure and to rebuild the same. In partially damaged pavement sections, patch repair works are carried out before constructing the overlay.

The maintenance engineers should therefore be vigilant and should take the decision in time for providing an overlay as and when needed.

10.5.2 Types of Overlay

The overlay combinations are divided into four categories based on the type of existing pavement and the overlay :

(i) Flexible overlay over flexible pavements

(ii) Cement concrete or rigid overlay over flexible pavements

(iii) Flexible overlays over cement concrete or rigid pavements

(iv) Cement concrete or rigid overlay over rigid pavements.

The choice of the overlay type depends upon number of factors including the thickness of overlay required, local materials, wheel loads cost etc.

10.5.3 Design of Overlay

The first stage of investigation before an overlay could be designed is evaluation of existing pavement as discussed in Art. 10.4. The overlay thickness required over a flexible pavement may be determined either by one of the conventional pavement design methods or by a non-destructive testing method like the Benkelman beam deflection method.

Flexible Overlay over Flexible Pavement by Conventional Design Method

The total pavement thickness requirement is designed for the design traffic and the existing conditions of subgrade. Any one of the design methods described in Art. 7.3 may be chosen for the design and the appropriate strength test is carried out on the sample of soil collected from the subgrade below the existing pavement. For example the CBR value or the subgrade soil is determined at the field density and the CBR method of pavement design as recommended by the IRC (Art. 7.3.3) is adopted for finding the total design thickness of the flexible pavement for the design traffic volume (Eq. 7.6 c). The existing thickness of the pavement is found from test pits dug along the wheelpath on the pavement. The overlay thickness required is given by the relation :

$$h_o = h_d - h_e \qquad (10.1)$$

where, h_o = overlay thickness required, cm

h_d = total design thickness required, presently determined, cm

h_e = total thickness of the existing pavement, cm

This is shown in Fig. 10.11. Suitable equivalency factors may be assigned to different types of materials in the pavement and overlay layers.

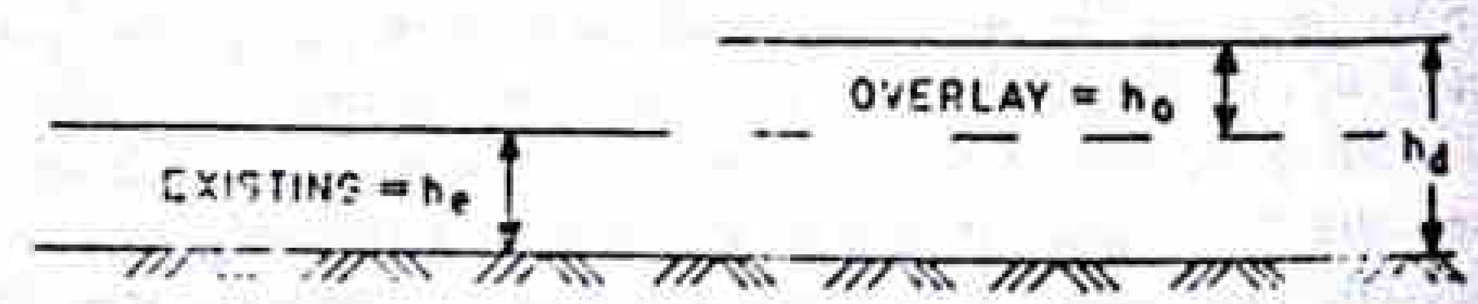


Fig. 10.11 Overlay Design

Overlay Design by Benkelman Beam Deflection Studies

Principle

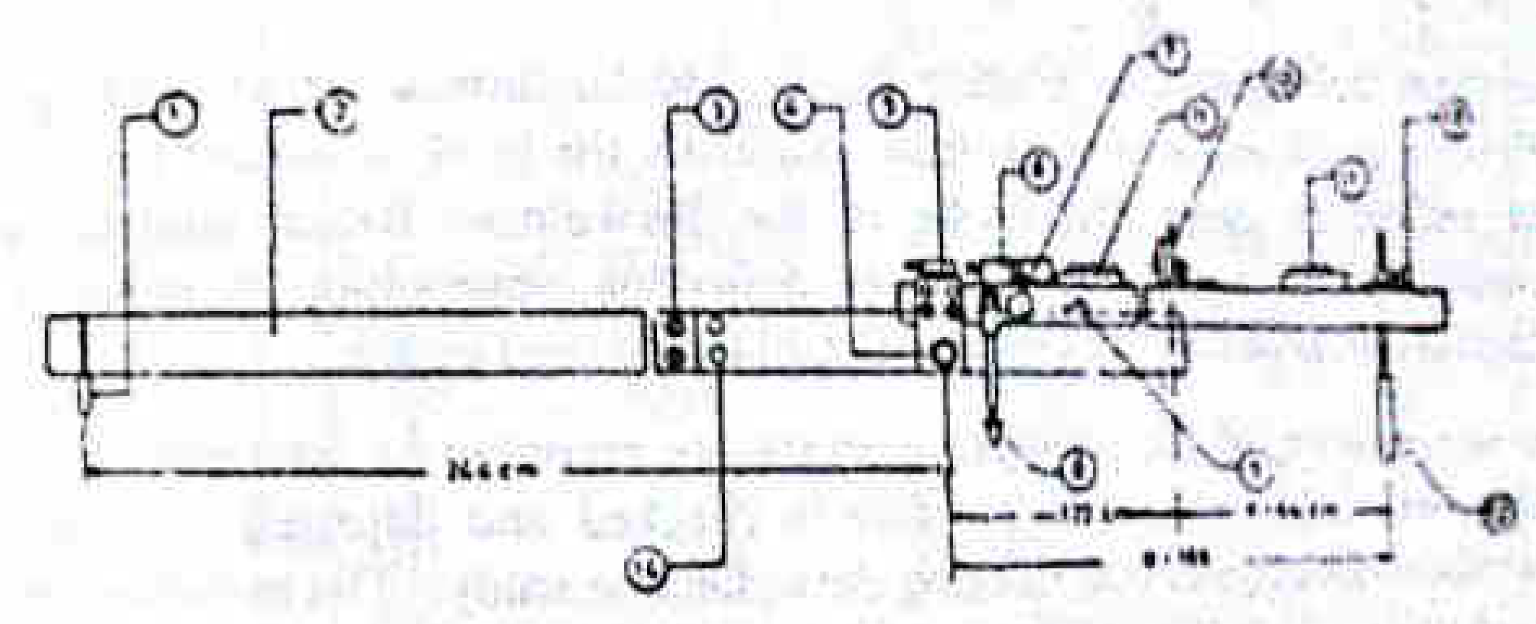
A well compacted pavement section or one which has been well conditioned by traffic deforms elastically under each wheel load application such that when the load moves away, there is an elastic recovery or rebound deflection of the deformed pavement surface. This is the basic principle of deflection method pavement evaluation or overlay design.

The maximum deflection under a design wheel load depends on several factors such as subgrade soil properties, moisture in the subgrade, pavement thickness and its composition, temperature of the pavement, loading particulars, etc. Therefore the amount

of pavement deflection under a design wheel load or its rebound deflection on removal of this load is a measure of the structural stability of the pavement system under the prevailing condition of the test. Larger rebound deflection indicates weaker pavement structure which may require earlier strengthening or higher overlay thickness. It is desirable to carry out the deflection studies soon after the monsoons when the pavement system may be at the weakest condition due to maximum subgrade moisture content.

Benkelman Beam

Benkelman Beam is a device which can be conveniently used to measure the rebound deflection of a pavement due to a dual wheel load assembly or the design wheel load. The equipment consists of a slender beam of length 3.66 m which is pivoted to a datum frame at a distance 2.44 m from the probe end. The datum frame rests on a pair of front leveling legs and a rear leg with adjustable height. The probe end of the beam is inserted between the dual rear wheels of truck and rests on the pavement surface at the centre of the loaded area of the dual wheel load assembly. A dial gauge is fixed on the datum frame with its spindle in contact with the other end of the beam in such a way that the distance between the probe end and the fulcrum of the beam is twice the distance between the fulcrum and the dial gauge spindle. (See Fig. 10.12). Thus the rebound deflection reading measured at the dial gauge is to be multiplied by two to get the actual movement of the probe end due to the rebound deflection of the pavement surface when the dual wheel load is moved forward. A loaded truck with rear axle load of 8170 kg is used for the deflection study. The design wheel load is a dual wheel load assembly of gross weight 4085 kg with an inflation pressure of 5.6 kg/cm^2.



(1) Probe (2) Beam (3) Counter sunk screw
(4) Beam support (5) Buzzer (6) Battery Box
(7) Push button switch (8) Levelling feet (9) Spirit Level
(10) Dial gauge (11) Lifting handles (12) Rear adjustable leg
(13) Lock screw (14) Round head screws

Fig. 10.12 Benkelman Beam

Procedure

The stretch of road length to be evaluated is first surveyed to assess the general condition of the pavement with respect to the ruts, cracks and undulations. Based on the above pavement condition survey, the pavement stretches are classified and grouped into different classes (of lengths not less than 500 metre) such as good, fair and poor for the purpose of Benkelman Beam deflection studies. The loading points on the pavement for deflection measurements are located along the wheel paths, on a line 0.9 metre from the pavement edge in the case of pavements of total width more than 3.5 m; the distance from the edge is reduced to 0.6 metre on narrower pavements. The number of loading points in a stretch and the spacing between them for the deflection measurements are to be decided depending on the objective of the project and the precision desired. A minimum of 10 deflection observations (preferably 20 observations) may be taken on each of the selected stretch of pavement. The deflection observation points may also be staggered if necessary and taken along the wheel paths on both the edges of the pavement. After marking the deflection observation points, the study is carried out in the following steps :

(i) The truck is driven slowly parallel to the edge and stopped such that the left side rear dual wheel is centrally placed over the first point for deflection measurement.

(ii) The probe end of the Benkelman Beam is inserted between the gap of the dual wheel and is placed exactly over the deflection observation point. See Fig. 10.13.

(iii) When the dial gauge reading is stationary or when the rate of change of pavement deflection is less than 0.025 mm per minute, the initial dial gauge reading D_0 is noted. Both the readings of the large and small needles of the dial gauge may be noted; the large needle may also be set to zero if necessary at this stage. (The least count of the large needle is 0.01 mm and that of small needle is 2.0 mm).

(iv) The truck is moved forward slowly through a distance of 2.7 m from the point and stopped. The intermediate dial gauge reading D_I is noted when the rate of recovery of the pavement is less than 0.025 mm per minute.

(v) The truck is then driven forward through a further distance of 9.0 m and the final dial gauge reading D_f is recorded as before.

(vi) The three deflection dial reading D_0, D_i and D_f form a set of readings at one deflection point under consideration. Similarly the truck is moved forward to the next deflection point, the brobe of the Benkelman Beam inserted and the procedure of noting the set of three deflection observation is repeated. The deflection observations are continued at all the desired points.

(vii) The temperature of the pavement surface are recorded at intervals of one hour during the study. The tyre pressure is checked and adjusted if necessary, at intervals of about three hours during the deflection study. The moisture content in the subgrade soil is also to be determined at suitable intervals.

(viii) The rebound deflection value D at any point is given by one of the following two conditions :

(a) If $D_i - D_f \leq 2.5$ divisions of the dial gauge or 0.025 mm, $D = 2 (D_0 - D_f)$ divisions of 0.01 mm units $= 0.02 (D_0 - D_f)$ mm

(b) If $D_i - D_f \geq 2.5$ division, this indicates that correction is needed for the vertical movement of the front legs. Therefore,

$D = 2 (D_0 - D_f) + 2 K (D_i - D_f)$ divisions.

The value of K is to be determined for every make of the Benkelman Beam and is given by the relation :

$$K = \frac{3d - 2e}{f}$$

where d = distance between the bearing of the beam and the rear adjusting leg

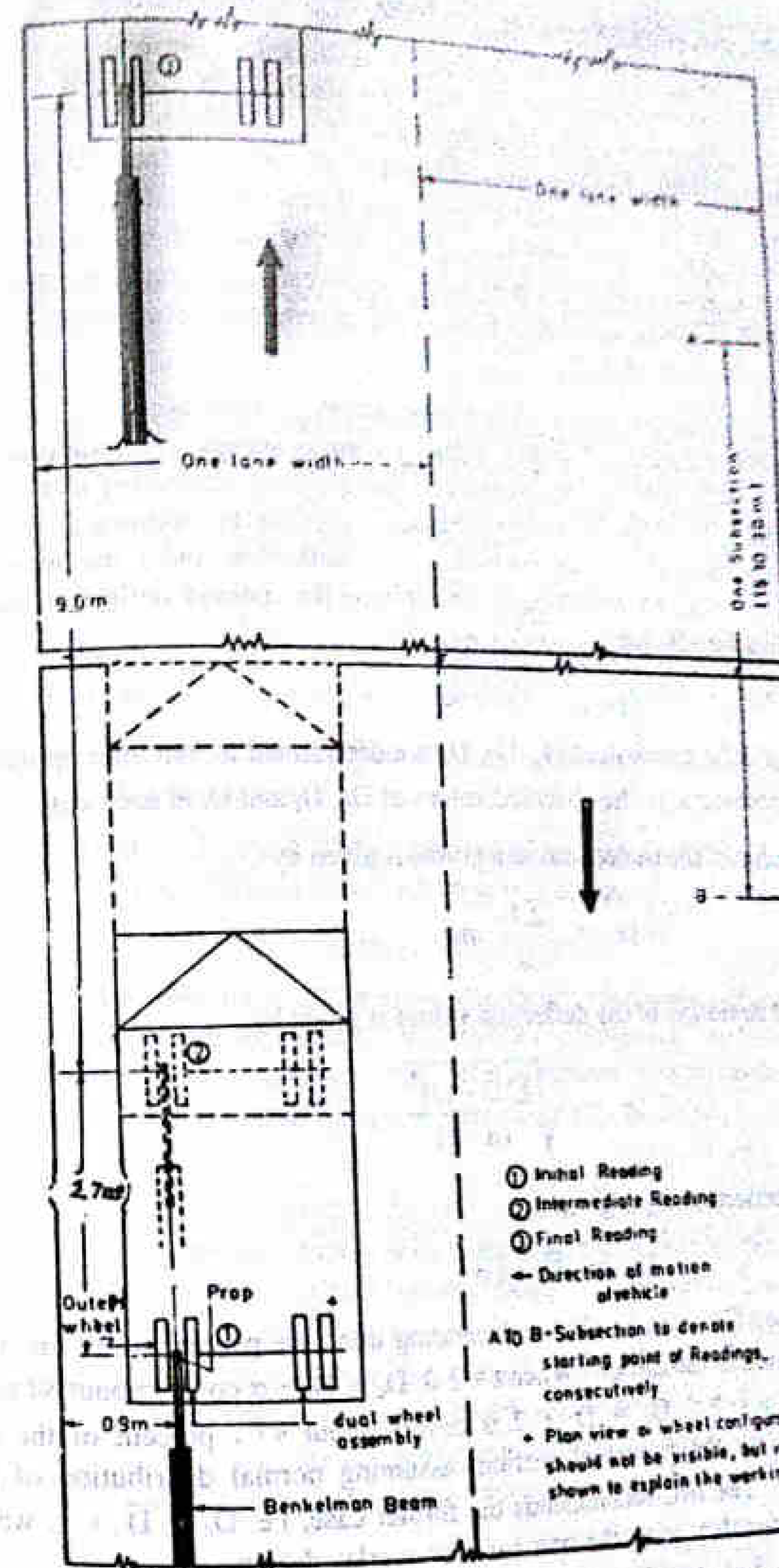


Fig. 10.13 Selection of Points and Procedure of Deflection Measurement

e = distance between the dial gauge and rear adjusting leg

f = distance between the front and rear legs.

The value of K of Benkelman Beam generally available in India is found to be 2.91. Therefore, the deflection value D in case (b) with leg correction is given by :

$$D = 0.02 (D_0 - D_f) + 0.0582 (D_i - D_f) \text{ mm}$$

Correction for pavement temperature and subgrade moisture variations

When the pavement consists of relatively thick bituminous layers like the bituminous macadam or asphaltic concrete in the base/binder/surface course, variations in temperature of pavement surface course cause variation in pavement deflection under the standard load. The IRC has suggested a standard pavement temperature of 35°C and a correction factor of 0.0065 mm per °C to be applied for the variation from this standard pavement temperature. The correction will be negative when the pavement temperature is above 35°C and positive when it is lower. However it is suggested that deflection studies should be carried out when the pavement temperature is above 30°C, if this correction factor is to be applied.

The seasonal variations cause variation in subgrade moisture. As it is always not possible to conduct deflection studies during monsoon season when subgrade moisture content is the highest, the IRC has suggested that tentative correction factors of 2.0 for clayey soils and 1.2 to 1.3 for sandy subgrade soils may be adopted if the deflection observations are made during dry seasons. The deflection under the worst subgrade moisture may therefore be estimated by multiplying the summer deflection value by the appropriate correction factor.

Analysis of Data

The rebound deflection values D_1, D_2, D_3 are determined in mm after applying the leg corrections, if necessary, to the observed values of D_o, D_f and D_i in each case.

The mean value of the deflections at n points is given by :

$$\overline{D} = \frac{\Sigma D}{n} \text{ mm}$$

The standard deviation of the deflection values is given by :

$$\sigma = \sqrt{\frac{\Sigma(\overline{D} - D)^2}{(n-1)}} \tag{10.2}$$

Characteristics deflection D_c is given by :

$$D_c = \overline{D} + t\,\sigma \tag{10.3}$$

Here the value of 't' is to be chosen depending upon the percentage of the deflection values to be covered in the design. When t = 1.0, $D_c = \overline{D} + \sigma$ covers about 84 percent of the cases; when t = 2.0, $D_c = \overline{D} + 2\sigma$ covers about 97.7 percent of the cases of deflection values on the pavement section, assuming normal distribution of rebound deflection values. The IRC recommends the former case, i.e. $D_c = \overline{D} + \sigma$, whereas in many other countries they adopt the later case for overlay design.

The necessary corrections for pavement temperature and subgrade moisture may be applied to the characteristic deflection value, D_c before designing the overlay thickness.

Overlay Thickness Design

The overlay thickness required h_o may be determined after deciding the allowable deflection D_a in the pavement under the design load. According to Ruiz's equation, overlay thickness h_o in cm is given by :

$$h_o = \frac{R}{0.434} \log_{10} \frac{D_c}{D_a} \text{ cm} \tag{10.4}$$

where h_o = thickness of bituminous overlay in cm

R = deflection reduction factor depending on the overlay material (usual values for bituminous overlays range from 10 to 15, the average values that may be generally taken being 12).

D_a = allowable deflection which depends upon the pavement type and the desired design life, values ranging from 0.75 to 1.25 mm are generally used in flexible pavements for overlay design.

The Indian Road Congress suggests the following formula for the design of overlay thickness equivalent to granular material of WBM layer. When superior materials are used in the overlay layer, the thickness value has to be suitably decreased taking 'equivalency factor' of the material into consideration.

$$h_o = 550 \log_{10} \frac{D_c}{D_a} \text{ mm} \tag{10.5}$$

where, h_o = thickness of granular or WBM overlay in mm

D_c = $(\overline{D} + \sigma)$, after applying the corrections for pavement temperature and subgrade moisture

D_a = 1.00, 1.25 and 1.5 mm, if the projected design traffic A is 1500 to 4500, 450 to 1500 and 150 to 450 respectively. Here,

$$A = \text{design traffic} = P\,[1 + r]^{(n+10)} \quad \text{(See Eq. 7.6c)}$$

When bituminous concrete or Bituminous Macadam with bituminous surface course is provided as the overlay, an equivalency factor of 2.0 is suggested by the IRC to decide the actual overlay thickness required. Thus, the thickness of bituminous concrete overlay in mm will be $h_o/2$ when the value of h_o is determined from Eq. 10.5.

Example 10.1

Benkelman Beam deflection studies were carried out on 15 selected points on a stretch of flexible pavement during summer season using a dual wheel load of 4085 kg, 5.6 kg/cm^2 pressure. The deflection values obtained in mm after making the necessary lag corrections are given below. If the present traffic consists of 750 commercial vehicles per day, determine the thickness of bituminous overlay required, if the pavement temperature during the test was 30°C and the correction factor for subsequent increase in subgrade moisture content is 1.3. Assume annual rate of growth of traffic as 7.5%. Adopt IRC guidances.

1.40, 1.32, 1.25, 1.35, 1.48, 1.60, 1.65, 1.55, 1.45, 1.40, 1.36, 1.46, 1.50, 1.52, 1.45 mm

Solution

$$\text{Mean deflection } \overline{D} = \frac{\Sigma D}{n} = \frac{21.74}{15} = 1.45 \text{ mm}$$

$$\text{Standard deviation} = \sqrt{\frac{\Sigma(\overline{D} - D)^2}{n-1}} = 0.107 \text{ mm}$$

Characteristics deflection $D_c = \bar{D} + \sigma = 1.557$ mm

Deflection after temp. correction $= 1.557 - (39 - 35)\ 0.0065 = 1.531$ mm

Corrected deflection for subgrade moisture $= 1.531 \times 1.3 = 1.99$ mm

Assuming the number of years after the last traffic count before the construction of overlay as n = 2, design traffic A is given by :

$$A = P\,[1 + r]^{(n+10)}$$

$$= 750\,[1 + 0.075]^{12} = 1768 \text{ com. veh./day}$$

Therefore as per IRC recommendations adopt allowable deflection $D_a = 1.0$ mm.

Overlay thickness of granular material

$$h_o = 550 \log_{10} \frac{D_c}{D_a} \text{ mm}$$

$$= 550 \log \frac{1.990}{1.00} = 164.4 \text{ mm} = 16.5 \text{ cm}$$

Assuming an equivalency factor of 2.0 for bituminous concrete overlay, the design thickness of bituminous concrete overlay

$$= 16.5 \times ½ = 8.25 \text{ cm}$$

Rigid Overlay Over Rigid Pavement

When a rigid or CC overlay is constructed over an existing rigid or CC pavement, the interface between the old and new concrete cannot have perfect bond such that the two slabs could act as a monolithic one. Two typical types of interface are possible : (i) providing maximum possible interface bond by making the old surface rough (ii) separating the two slabs at the interface by thin layer of bituminous material, or without interface bond.

For calculating the rigid overlay thickness, the existing rigid pavement is evaluated and the thickness for the design wheel load is calculated using the criterion laid down in article 7.4. To obtain the overlay thickness, the following relationship may be used :

$$h_o = (h_d^{\,a} - X h_e^{\,b})^n \qquad (10.6)$$

Here, h_o = rigid overlay thickness

h_d = design thickness

h_e = existing pavement thickness

Values of a, b, X and n depend upon the pavement and the method of overlay construction. Table 10.2 shows the recommended values of these factors.

Flexible Overlay Over Rigid Pavement

A flexible or bituminous overlay when provided over a rigid pavement, the wheel load is distributed through a larger area by the overlay, thus slightly reducing the wheel load stress on the old rigid pavement. Further the maximum temperature differential in the

Table 10.2 Rigid Overlay Design Factors

Agency	Existing pavement condition	X		
Crops of Engineers and, PCA	(i) Good condition	1.00		
	(ii) Initial cracking	0.75		
	(iii) Badly cracked	0.35		
Agency	**Construction method**	**a**	**b**	**n**
(i) Crops of Engineers	(i) Poured directly on old pavement	1.40	1.40	1/1.4
	(ii) Levelling course	2.20	2.00	1/3
(ii) PCA	(i) as (i) above	1.87	2.00	1/2
	(ii) as (ii) above	2.00	2.00	1/2

rigid pavement is also decreased due to the bituminous overlay, thus causing a substantial reduction in the warping stress and also in the maximum combined stress. Thus a rigid or CC pavement may be Strengthened by a bituminous overlay before the pavement develops structural crack and starts failing. The useful life of the rigid pavement may be increased considerably by a suitably designed and constructed bituminous overlay placed at the right time.

For calculating the thickness of flexible overlay over rigid pavements the following relationship is employed.

$$h_f = 2.5\,(F h_d - h_e) \qquad (10.7)$$

Here h_f = flexible overlay thickness

h_e = existing rigid pavement thickness

h_d = design thickness of rigid pavement

F = factor which depends upon modulus of existing pavement

For calculating the thickness of bituminous overlay, the following relationship is used.

$$h_b = \frac{h_f}{1.5} \qquad (10.8)$$

i.e., $h_b = 1.66\,(F h_d - h_e)$

Rigid Overlay Over Flexible Pavement

The thickness of rigid overlay is calculated by using the design criteria for rigid pavements as laid down in Article 7.4. The plate bearing test is conducted on the existing flexible pavement and K value is thus obtained. The design is made for this K value and the design wheel load.

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PROBLEMS

1. Discuss briefly the importance of highway maintenance.
2. (a) What are the general causes of pavement failures ?

   (b) Write a note on Maintenance Management System.
3. Explain the procedure for patch repair works in :

   (a) WBM pavement

   (b) Bituminous pavement

   (c) Bituminous pavement during monsoons (when the pavement is wet).
4. What are the various types of failures in flexible pavement ? Explain the causes.

5. Explain the various type of failures in cement concrete pavements and their causes.
6. Explain how the maintenance of the following pavements are carried out.

   (a) Earth roads

   (b) Bituminous surfaces

   (c) Cement concrete pavements
7. What are the various causes of formation of waves and corrugations in flexible pavements ? Suggest remedial measures.
8. Explain the necessity of design approach and method of strengthening of existing pavements for the following cases.

   (a) Flexible overlay over flexible pavement

   (b) Flexible overlay over rigid pavement

   (c) Rigid overlay over flexible pavement

   (d) Rigid overlay over rigid pavement
9. Write short notes on :

   (a) Map (alligator) cracking

   (b) Reflection cracking

   (c) Skidding of pavement surfaces

   (d) Scaling of cement concrete

   (e) Mud pumping

   (f) Spalling of joints.
10. Write a descriptive note on pavement evaluation.
11. Explain the principle and uses of Benkelman Beam test.
12. Existing black top pavement was tested using Benkelman Beam with a test vehicle of 8170 kg rear axle load. Observations recorded at a pavement temperature of 43°C are given below :

    Length of test stretch = 300 m

Serial number subsection	Rebound deflection (mm)	Serial number subsection	Rebound deflection (mm)
1	1.46	7	1.68
2	1.52	8	1.74
3	1.56	9	1.96
4	1.76	10	1.42
5	1.96	11	1.56
6	1.74	12	1.62

Compute the thickness of overlay of bituminous concrete, taking allowable deflection as 1.25 mm, if the factor for subgrade moisture correction is 2.0.

Chapter 11

Highway Drainage

11.1 INTRODUCTION

Highway drainage is the process of removing and controlling excess surface and sub-soil water within the right of way. This includes interception and diversion of water from the road surface and subgrade. The installation of suitable surface and sub-surface drainage system is an essential part of highway design and construction.

During rains, part of the rain water flows on surface and part of it percolates through the soil mass as gravitational water until it reaches the ground water below the water table. Removal and diversion of surface water from the roadway and adjoining land is termed as *surface drainage*. Diversion or removal of excess soil-water from the subgrade is termed as *sub-surface drainage*. Some water is retained in the pores of the soil mass and on the surface of soil particles by surface tension and adsorptive forces, which cannot be drained off by normal gravitational methods and this water is termed as *held water*.

11.2 IMPORTANCE OF HIGHWAY DRAINAGE

11.2.1 Significance of Drainage

An increase in moisture content causes decrease in strength or stability of a soil mass; the variation in soil strength with moisture content also depends on the soil type and the mode of stress application. Highway drainage is important because of the following reasons :

(i) Excess moisture in soil subgrade causes considerable lowering of its stability. The pavement is likely to fail due to subgrade failure as discussed in Article 10.1.

(ii) Increase in moisture cause reduction in strength of many pavement materials like stabilized soil and water bound macadam.

(iii) In some clayey soils variation in moisture content causes considerable variation in volume of subgrade. This sometimes contributes to pavement failure.

(iv) One of the most important causes of pavement failure by the formation of waves and corrugations in flexible pavements is due to poor drainage.

(v) Sustained contact of water with bituminous pavements causes failures due to stripping of bitumen from aggregates like loosening or detachment of some of the bituminous pavement layers and formation of pot holes.

(vi) The prime cause of failures in rigid pavements by mud pumping is due to the presence of water in fine subgrade soil.

(vii) Excess water on shoulders and pavement edge causes considerable damage.

(viii) Excess moisture causes increase in weight and thus increase in stress and simultaneous reduction in strength of the soil mass. This is one of the main reasons of failure of earth slopes and embankment foundations.

(ix) In places where freezing temperatures are prevalent in winter, the presence of water in the subgrade and a continuous supply of water from the ground water can cause considerable damage to the pavement due to *frost action*.

(x) Erosion of soil from top of unsurfaced roads and slopes of embankment, cut and hill side is also due to surface water.

11.2.2 Requirements of Highway Drainage System

(i) The surface water from the carriageway and shoulder should effectively be drained off without allowing it to percolate to subgrade.

(ii) The surface water from the adjoining land should be prevented from entering the roadway.

(iii) The side drain should have sufficient capacity and longitudinal slope to carry away all the surface water collected.

(iv) Flow of surface water across the road and shoulders and along slopes should not cause formation of cross ruts or erosion.

(v) Seepage and other sources of under ground water should be drained off by the subsurface drainage system.

(vi) Highest level of ground water table should be kept well below the level of subgrade, preferably by atleast 1.2 m.

(vii) In waterlogged areas special precautions should be taken, especially if detrimental salts are present or if flooding is likely to occur.

11.3 SURFACE DRAINAGE

The surface water is to be collected and then disposed off. The water is first collected in longitudinal drains, generally in side drains and then the water is disposed off at the nearest stream, valley or water course. Cross drainage structures like culverts and small bridges may be necessary for the disposal of surface water from the road side drains.

11.3.1 Collection of Surface Water

The water from the pavement surface is removed by providing the camber or cross slope to the pavement. The rate of this cross slope is decided based on type of pavement surface and amount rainfall. The recommended range of camber for different types of pavement surfaces are given in Article 4.1.2.

In rural highways, the water which is drained from the pavement surface has also to drain across the shoulders before it is lead to the side drains. Hence the shoulders of these roads are constructed with suitable cross slope so that the water is drained across the shoulders to the side drains. These side drains of rural roads are generally open kutcha (unlined) drains of trapezoidal shape, cut to suitable cross section and longitudinal slopes. These side drains are provided parallel to the road alignment and hence these are also called longitudinal drains. In embankments the longitudinal drains are provided on one or both sides beyond the toe; in cuttings, drains are installed on either side of the formation. See Fig. 4.6 to 4.8. But in places where there is restriction of space, construction of deep open drains may be undesirable. This is particularly true when the road formation is in cutting. In such cases covered drains or drainage trenches properly filled with layers of coarse sand and gravel may be used. See Fig. 11.1.

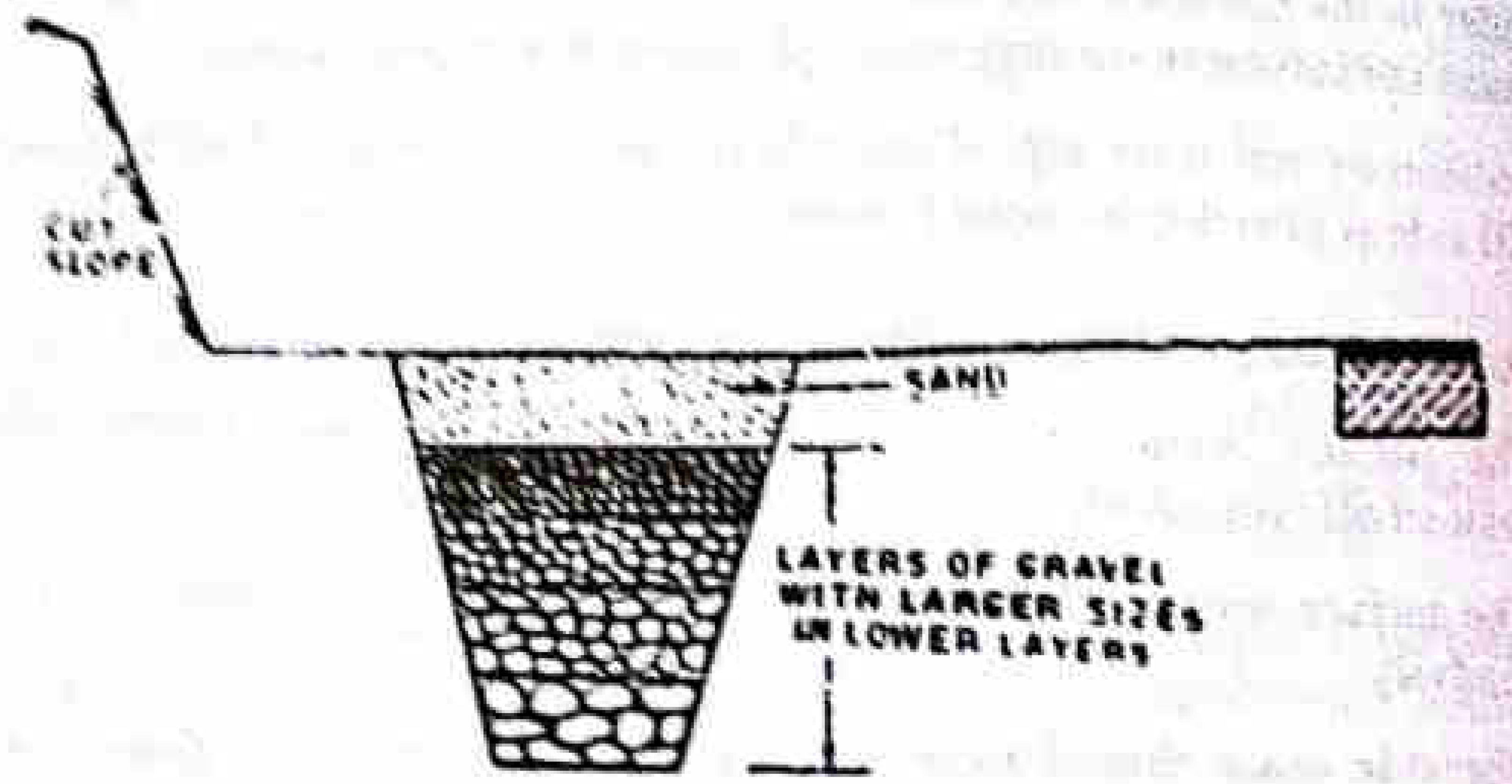

Fig. 11.1 Covered Drainage Trenches

In urban roads because of the limitation of land width and also due to the presence of foot path, dividing islands and other road facilities, it is necessary to provide under ground longitudinal drains. Water drained from the pavement surface can be carried forward in the longitudinal direction between the kerb and the pavement for short distances (See Fig. 4.9). This water may be collected in catch pits at suitable intervals and lead through under ground drainage pipes. Section of a typical catch pit with grating to prevent the entry of rubbish into the drainage system is shown in Fig. 11.2.

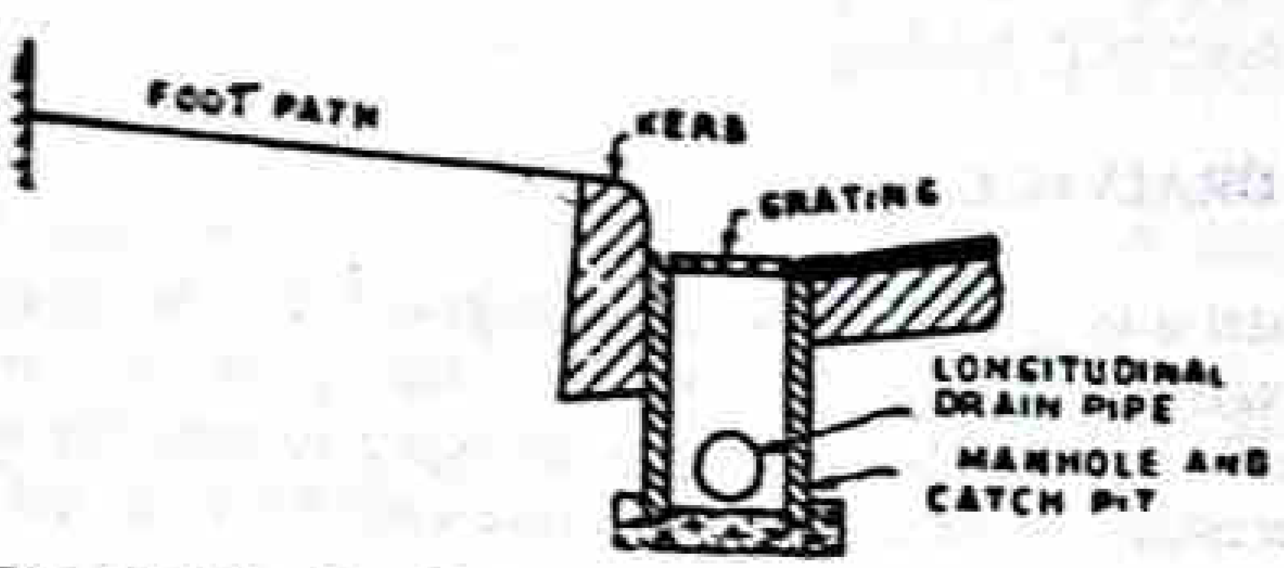

Fig. 11.2 Surface Drainage System in Urban Roads

Drainage of surface water is all the more important in hill roads. Apart from the drainage of water from the road formation, the efficient diversion and disposal of water flowing down the hill slope across the road and that from numerous cross streams is an important part of hill road construction. If the drainage system in hill road is not adequate and efficient, it will result in complex maintenance problems. The details of hill road drainage are given in Article 12.5.

11.3.2 Design of Surface Drainage System

The design of surface drainage system may be divided into two phases :

(i) Hydrologic analysis

(ii) Hydraulic analysis

Hydrologic Analysis

The main objective of hydrologic analysis is to estimate the maximum quantity of water expected to reach the element of the drainage system under consideration. A portion of the precipitation during the rain fall infiltrates into the ground as ground water and a small portion gets evaporated. The remaining portion of water which flows over the surface is termed as run-off. Various factors affecting the run-off are rate of rainfall, type of soil and moisture condition, topography of the area, type of ground cover like vegetation etc. The surface drainage system is to be designed to drain away the surface run-off water. The details of rain fall in the area including intensity duration and frequency of occurrence of storm are to be collected. Next the run-off and maximum rate of run-off for the area under consideration is determined using any of the accepted approaches. It is also necessary to find the drainage area from where water is likely to flow in.

Rational formula is widely used to estimate the peak run-off water for highway drainage. The rational formula, in its simplest form is given by :

$$Q = C\, i\, A_d \qquad (11.1)$$

where Q = run-off, m^3/sec

C = run-off coefficient, expressed as a ratio of run-off to rate of rain fall

i = intensity of rain fall, mm/sec

A_d = drainage area in 1000 m^2

The above expression is dimensionally not balanced. The value of run-off coefficient C depends mainly on the type of surface and its slope. The C-values may be taken as 0.8 to 0.9 for bituminous and cement concrete pavements, 0.35 to 0.70 for gravel and WBM pavements, 0.40 to 0.65 for impervious soil, 0.30 to 0.55 for soil covered with turf and 0.05 to 0.30 for pervious soils. When the drainage area A_d consists of several types of surfaces with run-off coefficients C_1, C_2, C_3, with their respective areas A_1, A_2, A_3, the weighted value of run-off coefficient C is determined from :

$$C = \frac{A_1C_1 + A_2C_2 + A_3C_3 + ...}{A_1 + A_2 + A_3 ...} \qquad (11.2)$$

The design value of the rain fall intensity i is to be determined for the expected duration of storm and frequency of occurrence. Therefore the inlet time for the storm water to flow from the remotest point in drainage area to the drain inlet is estimated using the chart (see Fig. 11.3). The time for water to flow through the drain between the inlet and outlet points is determined based on the allowable velocity of flow in the drain, generally ranging from 0.3 to 1.5 m/sec, depending on soil type. The time of concentration or the duration of storm for design may be taken as the sum of inlet time and the time of flow through the drain. The frequency of occurrence of the storm or the

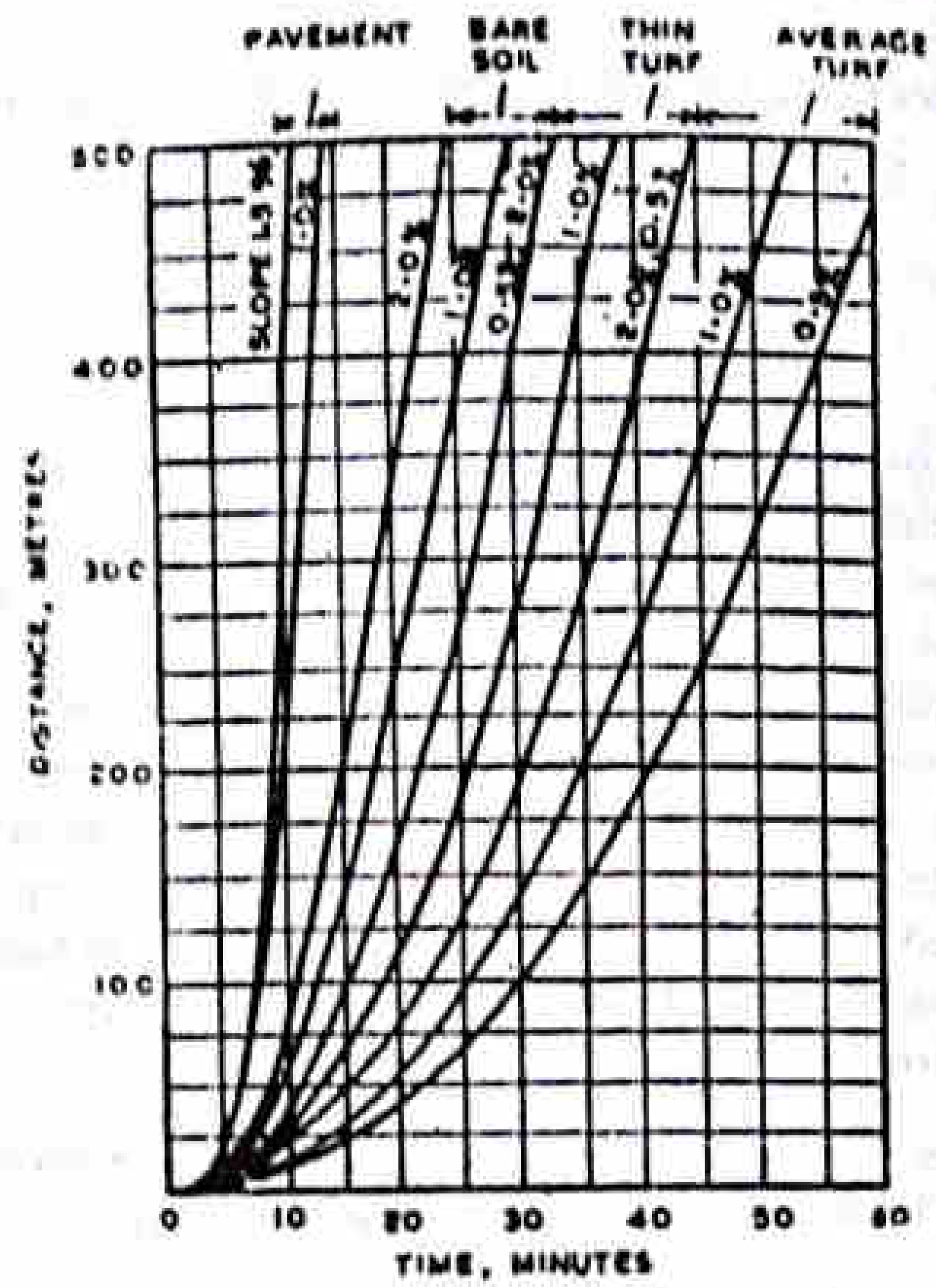

Fig. 11.3 Time of Flow to Inlet

return period (may be taken as 5, 10, 25 or 50 years). From the chart (Fig. 11.4) the design value of the rainfall intensity i is found corresponding to the duration of storm and the selected value of frequency.

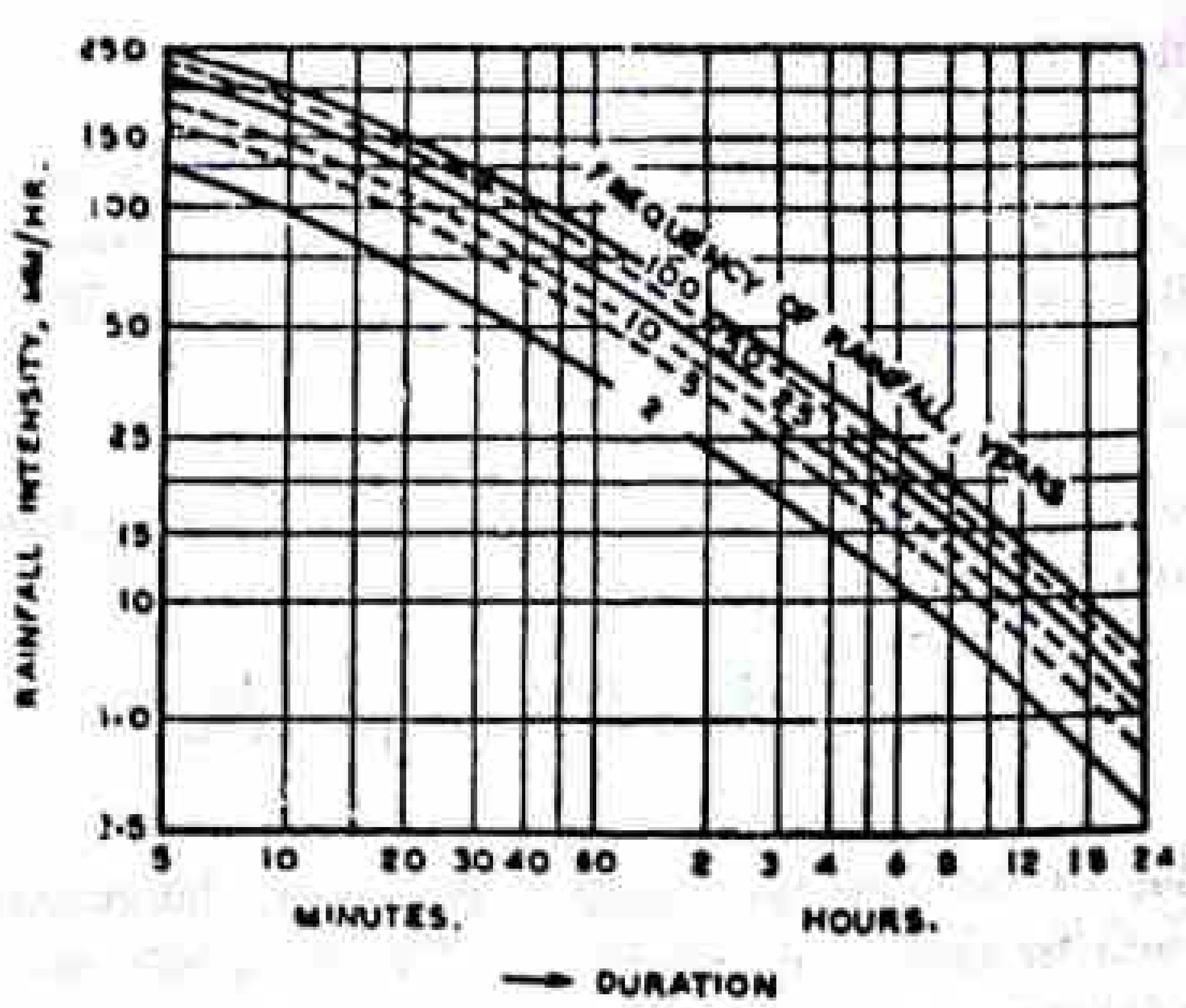

Fig. 11.4 Typical Rainfall Intensity Duration Curve

The drainage area from which the surface water is expected to flow to a side drain is determined with the aid of a contour map or by studying the topography of the drainage area. This area is expressed in units of 1000 square metre to obtain the value of A_d to be used in Eq. 11.1 Thus with the estimated value of C, i and A_d the design value of run-off Q for the longitudinal side drain is determined.

Hydraulic Design

Once the design runoff Q is determined, the next step is the hydraulic design of drains. The side drains and partially filled culverts are designed based on the principles of flow through open channels.

If Q is the quantity of surface water (m^3/sec) to be removed by a side drain and V is the allowable velocity of flow (m/sec) on the side drain, the area of cross section A of the channel (m^2) is found from the relation :

$$Q = A V \tag{11.3}$$

The velocity of unlined channel must be high enough to prevent silting and it should not be too high as to cause erosion. The allowable velocity of flow depends on the soil type; for sand and silt it is 0.3 to 0.5 m/sec, loam 0.6 to 0.9, clay 0.9 to 1.5 and gravel 1.2 to 1.5 m sec. For good soil covered with well established grass 1.5 to 1.8 m/sec may be allowed. By adjusting the value of slope S in Eq. 11.4, it is possible to limit the velocity of flow, V within permissible limit. Assuming uniform and steady flow through channel of uniform cross section and slope, *Manning's* formula is used for determining the velocity of flow or the longitudinal slope which is given by :

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{11.4}$$

Here, V = average velocity, m/sec.

n = Manning's roughness coefficient

R = hydraulic radius m (cross section area of flow divided by wetted perimeter)

S = longitudinal slope of channel

The *roughness coefficient* values depend on the type of soil in unlined channels. For ordinary earth, the value of n = 0.02, whereas for earth with heavy vegetation or grass the value of n = 0.05 to 0.1. in lined channels, the roughness coefficient depends on the type of lining. For well finished concrete, the value of n = 0.013 but for rough rubble and riprap, n = 0.04.

The slope S of the longitudinal drain of a known or an assumed cross section and depth of flow, may be determined using Manning's formula (Eq. 11.4) for the design values of velocity of flow V, roughness coefficient n and hydraulic radius R.

Data for Drainage Design

The following data are to be collected for the design of road side drain :

(i) Total road length and width of land from where water is expected to flow on the stretch of the side drain.

(ii) Run-off coefficients of different types of surfaces in the drainage area and their respective areas (such as paved area, road shoulder area, turf surface, etc.)

(iii) Distance from farthest point in the drainage area to the inlet of the side drain along the steepest gradient and the average value of the slope.

(iv) Type of soil of the side drain, Roughness coefficient allowable velocity of flow in the drain.

(v) Rain fall data including average intensity and frequency of recurrence of flood.

Design Steps

Simplified steps for the design of longitudinal drains of a road to drain off the surface water are given below :

(i) The frequency of return period such as 10 years, 25 years etc. is decided based on finances available and desired margin of safety, for the design of the drainage system.

(ii) The values of coefficients of run-off C_1, C_2, C_3 etc. from drainage areas A_1, A_2, A_3 etc. are found and the weighted value of C is computed.

(iii) Inlet time T_1 for the flow of storm water from the farthest point in the drainage area to the drain inlet along the steepest path of flow is estimated from the distance, slope of the ground and type of the cover. Figure 11.3 may be used for this purpose.

(iv) Time of flow along the longitudinal drain T_2 is determined for the estimated length of longitudinal drain L upto the nearest cross drainage or a water course, and for the allowable velocity of flow V in the drain i.e., $T_2 = \frac{L}{V}$.

(v) The total time T for inlet flow and flow along the drain is taken as the time of concentration or the design value of rain fall duration, $T = T_1 + T_2$.

(vi) From the rain fall intensity-duration-frequency curves (Fig. 11.4) the rain fall intensity i is found in mm/sec. corresponding to duration T and frequency of return period.

(vii) The total area of drainage A_d is found in units of 1000 m^2.

(viii) The run-off quantity Q is computed = C i A_d.

(ix) The cross sectional area of flow A of the drain is calculated = Q/V, where V is the allowable speed of flow in the drain.

(x) The required depth of flow in the drain is calculated for a convenient bottom width and side slop of the drain. The actual depth of the open channel drain may be increased slightly to give a free board. The hydraulic mean radius of flow R is determined.

(xi) The required longitudinal slope S of the drain is calculated using Manning's formula adopting suitable value of roughness coefficient n.

Example 11.1

The distance between the farthest point in the turf covered drainage area (with an average slope of 1.5 % towards the drain) and the point of entry to side drain is 200 m. The weighted average value of the run-off coefficient is 0.25. The length of the longitudinal open drain in a sandy clay soil from the inlet point to the cross drainage is 540 m. The velocity of flow in the side drain may be assumed as 0.6 m/sec so that silting and erosion are prevented. Estimate the design quantity of flow on the side drain for a ten-years period of frequency of occurrence of the storm.

Solution

$C = 0.25$

Inlet time T_1 (Fig. 11.3) for average turf with 1.5% slope corresponding to 200 m distance = 33 mins. (by interpolation)

Time T_2 for water to flow through 540 m length of drain at 0.6 m/sec

$$= \frac{540}{0.6 \times 60} = 15 \text{ mins.}$$

Duration or time of concentration $T = 33 + 15 = 48$ mins.

Drainage area $540 \times 200 = 108000\ m^2$.

$A_d = 108000/1000 = 108$

Design value of rain fall intensity for 10 year frequency of occurrence and corresponding to 48 mins. Duration (Fig. 11.4) = 70 mm/hr, therefore, i = 70/3600 mm/sec.

Design quantity of flow, $Q = C\,i\,A$

$$= 0.25 \times \frac{70}{3600} \times 108 = 0.525\ m^3/sec.$$

Example 11.2

The maximum quantity of water expected in one of the open longitudinal drains on clayey soil is 0.9 m^3/sec. Design the cross section and longitudinal slope of trapezoidal drain assuming the bottom width of the trapezoidal section to be 1.0 m and cross slope to be 1.0 vertical to 1.5 horizontal. The allowable velocity of flow in the drain is 1.2 m/sec and Manning's roughness coefficient is 0.02.

Solution

(i) Cross Section

The allowable velocity of flow through the clay soil V = 1.2 m/sec.

From Eq. 11.3, cross section area of drain $A = Q/V = 0.9/1.2 = 0.75\ m^2$

For the trapezoidal section with bottom width 1.0 m and side slope 1.0 vertical to 1.5 horizontal, when the depth of flow is d metre, the top width would be (1 + 3d) and the cross section area of the drain $= (1 + 1 + 3d)\,d/2 = d + \frac{3d^2}{2}$.

This area has been found to be 0.75 m^2 (in the previous step)

Therefore,

$$d + \frac{3d^2}{2} = 0.75$$

i.e.,

$$1.5\,d^2 + d - 0.75 = 0$$

Solving this quadratic equation for d,

$$d = \frac{-1 \pm \sqrt{1^2 - 4 \times 1.5(-0.75)}}{2 \times 1.5} = 0.45 \text{ m}$$

This is the actual depth of flow for the design quantity of water through the trapezoidal section. Therefore, allowing a free board of 0.15 m, the depth of the side drain may be taken as 0.45 + 0.15 = 0.6 m.

(ii) Slope

The longitudinal slope may be found using Manning's formula (Equation 11.4).

$$V = \frac{1}{n} . R^{2/3} . S^{1/2}$$

Assume roughness coefficient n for clay = 0.02 and velocity of flow V = 1.2 m/sec. For the assumed trapezoidal section, the wetted area of cross section is 0.75 m^2 and the wetted perimeter is = $\sqrt{0.45^2 + (1.5 \times 0.45)^2} \times 2 + 1.0 = 2.62$ m.

$$\text{Hydraulic radius } R = \text{area/perimeter} = 0.75/2.62 = 0.286$$

$$S^{1/2} = \frac{Vn}{R^{2/3}} = \frac{1.2 \times 0.02}{(0.286)^{2/3}} = 0.0553$$

$$\text{Slope } S = 0.0031 \text{ or } 1 \text{ in } 322.5$$

Therefore, provide a longitudinal slope of 1 in 320.

Example 11.3

The surface water from road side is drained to the longitudinal side drain from across one half a bituminous pavement surface of total width 7.0 m, shoulder and adjoining land of width 8.0 m on one side of the drain. On the other side of the longitudinal drain, water flows across from reserve land with grass and 2% cross slope towards the side drain, the width of this strip of land being 25 m. The run off coefficients of the pavement, shoulder and reserve land with grass surface are 0.8, 0.25 and 0.35 respectively. The length of the stretch of land parallel to the road from where water is expected to flow to the side drain is about 400 m. Estimate the quantity of run-off flowing in the drain assuming 25 years period of frequency.

Design the cross section and slope of the side drain in loamy soil with Manning's roughness coefficient = 0.022 and suitable speed of flow = 0.8 m/sec.

Solution

(i) Quantity of Run-Off

Drainage area consists of

(1) pavement area = 3.5 × 400 = 1400 m^2 = A_1, with C_1 = 0.8

(2) area of shoulder and adjoining land = 8 × 400 = 3200 m^2 = A_2, with C_2 = 0.25 and

(3) area of land on the other side of the drain = 25 × 400 = 10000 m^2 = A_3 with C_3 = 0.35

$$\text{Total drainage area} = 1400 + 3200 + 10000 = 14600 \text{ m}^2$$

$$A_d = 14600/1000 = 14.6$$

Weighted value of run-off coefficient

$$C = \frac{A_1C_1 + A_2C_2 + A_3C_3}{A_1 + A_2 + A_3} = 0.371$$

The maximum distance of flow across the land upto longitudinal drain is 25 m along the reserve land with average turf and cross slope 2%. Therefore, inlet time T_1 from Fig. 11.3 = 11 min.

Time of flow T_2 along longitudinal drain of length 400 m on loamy soil with a speed of 0.8 m/sec = $\frac{400}{0.8 \times 60}$ = 8.33 min.

Total duration of rain fall = 11.00 + 8.33 = 19.33 min.

From Fig. 11.4, corresponding to 19.33 min. duration and 25 years period, rain fall intensity = 125 mm/hr.

$$i = 125/(60 \times 60) = 0.0347 \text{ mm/sec}$$

$$Q = C\, i\, A_d = 0.371 \times 0.0347 \times 14.6 = 0.188 \text{ m}^3/\text{sec}$$

(ii) Cross Section

Area of cross section of flow in the drain is given as

$$A = Q/V = 0.188/0.8 = 0.235 \text{ m}^2$$

Assuming bottom width of drain as 0.5 m, slope of 1.0 vertical to 1.5 horizontal and depth of flow as d, top width = (0.5 + 3 d) m.

Area of cross section of flow in drain is given by

$$0.235 = (0.5 + 0.5 + 3\,d)\,\frac{d}{2} = 0.5\,d + 1.5\,d^2$$

i.e., $1.5\,d^2 + 0.5\,d - 0.235 = 0$

Solving the quadratic equation,

$$d = \frac{-0.5 \pm \sqrt{0.5^2 - 4 \times 1.5(-0.235)}}{2 \times 1.5} = 0.263 \text{ m}$$

Therefore, the average depth of the drain may be taken as 0.40 m, after allowing a free board of about 14 cm.

(iii) Slope of Drain

When the depth of flow in the trapezoidal drain is 0.263 m, the slope sides of the trapezium is equal to 0.474 m.

$$\text{Wetted perimeter} = 0.5 + 2 \times 0.474 = 1.448 \text{ m}$$

$$R = \text{Area/wetted perimeter} = 0.235/1.448 = 0.162$$

$$V = \frac{R^{2/3}S^{1/2}}{n}$$

$$S^{1/2} = V \times n/R^{2/3} = \frac{0.8 \times 0.025}{(0.162)^{2/3}} = 0.0672$$

$$S = 0.00452, \text{ or } 1 \text{ in } 221$$

Therefore, provide a longitudinal slope of 1 in 220

11.3.3 Cross Drainage

Whenever streams have to cross the roadway, facility for cross drainage is to be provided. Also often the water from the side drain is taken across by these cross drain in order to divert the water away from the road, to a water course or valley. The cross drainage structures commonly in use are *culverts* and *small bridges*. When a small stream crosses a road with a linear waterway less than about six meter, the cross drainage structure provided is called culvert; for higher values of linear waterway, the structure is called a bridge.

The common types of culverts in use are

Slab culvert

Box culvert

Arch culvert

Pipe culvert

In slab culverts RCC slab is placed over *abutments* made of masonry and the span is generally limited to 3 metre. Box culvert of square or rectangular shapes is made of RCC. Arch culvert is generally built using brick or stone masonry, plain cement concrete may also be used. Pipe culverts of minimum diameter 75 cm and made of steel or prefabricated RCC is used when the discharge is low.

Various types of bridges are in use; the choice is based on several considerations including the span. RCC and steel bridges are commonly constructed these days.

On less important roads, in order to reduce the construction cost of cross drainage structures, sometimes submersible bridges or *cause ways* are constructed. During the floods the water will flow over the road. The total period interruption to traffic has however to be kept as low as possible, not exceeding about 15 days in a year.

11.4 SUB-SURFACE DRAINAGE

Changes in moisture content of subgrade are caused by fluctuations in ground water table seepage flow, percolation of rain water and movement of capillary water and even water vapour. In sub-surface drainage of highways, it is attempted to keep the variation of moisture in subgrade soil to a minimum. However only the *gravitational water* is drained by the usual drainage systems.

11.4.1 Lowering of Water Table

The highest level of water table should be fairly below the level of subgrade, in order that the subgrade and pavement layers are not subjected to excessive moisture. From practical considerations it is suggested that the water table should be kept atleast 1.0 to 1.2 m below the subgrade. In places where water table is high (almost at ground level at times) the best remedy is to take the road formation on embankment of height not less than 1.0 to 1.2 metre. When the formation is to be at or below the general ground level, it would be necessary to lower the water table.

If the soil is relatively permeable, it may be possible to lower the high water table by merely construction of longitudinal drainage trenches with drain pipe and filter sand. The depth of the trench would on the required lowering of water table, distance between the drainage trenches and soil types. See Fig. 11.5.

If the soil is relatively less permeable, the lowering of the ground water level may not be adequate at the centre of the pavement or in between the two longitudinal drainage trenches. Hence in addition, *transeverse drains* may have to be provided in order to effectively drain off the water and thus to lower the water table upto the level of transverse drains. A typical sketch of sub-surface drainage system with longitudinal and transverse drains for effective lowering of water table is shown in Fig. 11.6.

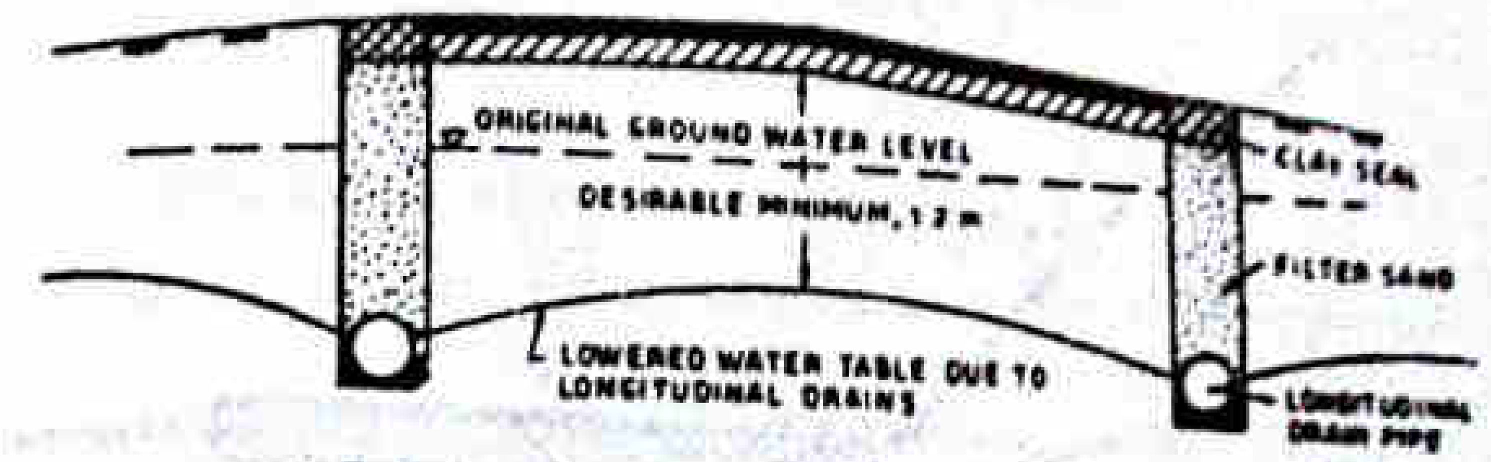

Fig. 11.5 Lowering High Water-table in Permeable Soils

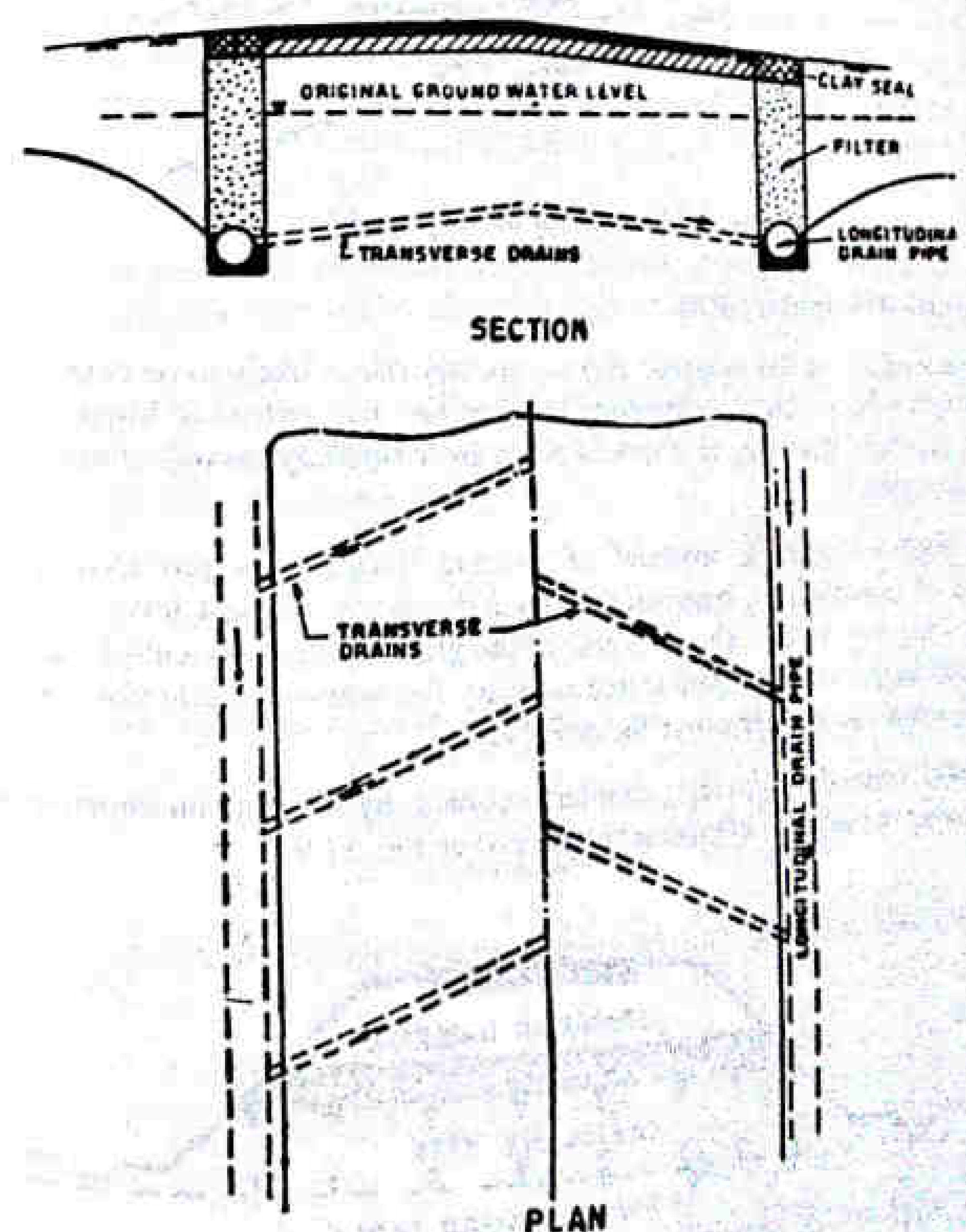

Fig. 11.6 Sub-surface Drainage System with Transverse Drains

11.4.2 Control of Seepage Flow

When the general ground as well as the impervious strata below are sloping, seepage flow is likely to exist. If the seepage zone is at depth less than 0.6 to 0.9 metre from the subgrade level, longitudinal pipe drain in trench filled with filter material and clay seal may be constructed to intercept the seepage flow. Figure 11.7 shows the method by which the seepage line can be lowered to the desired depth.

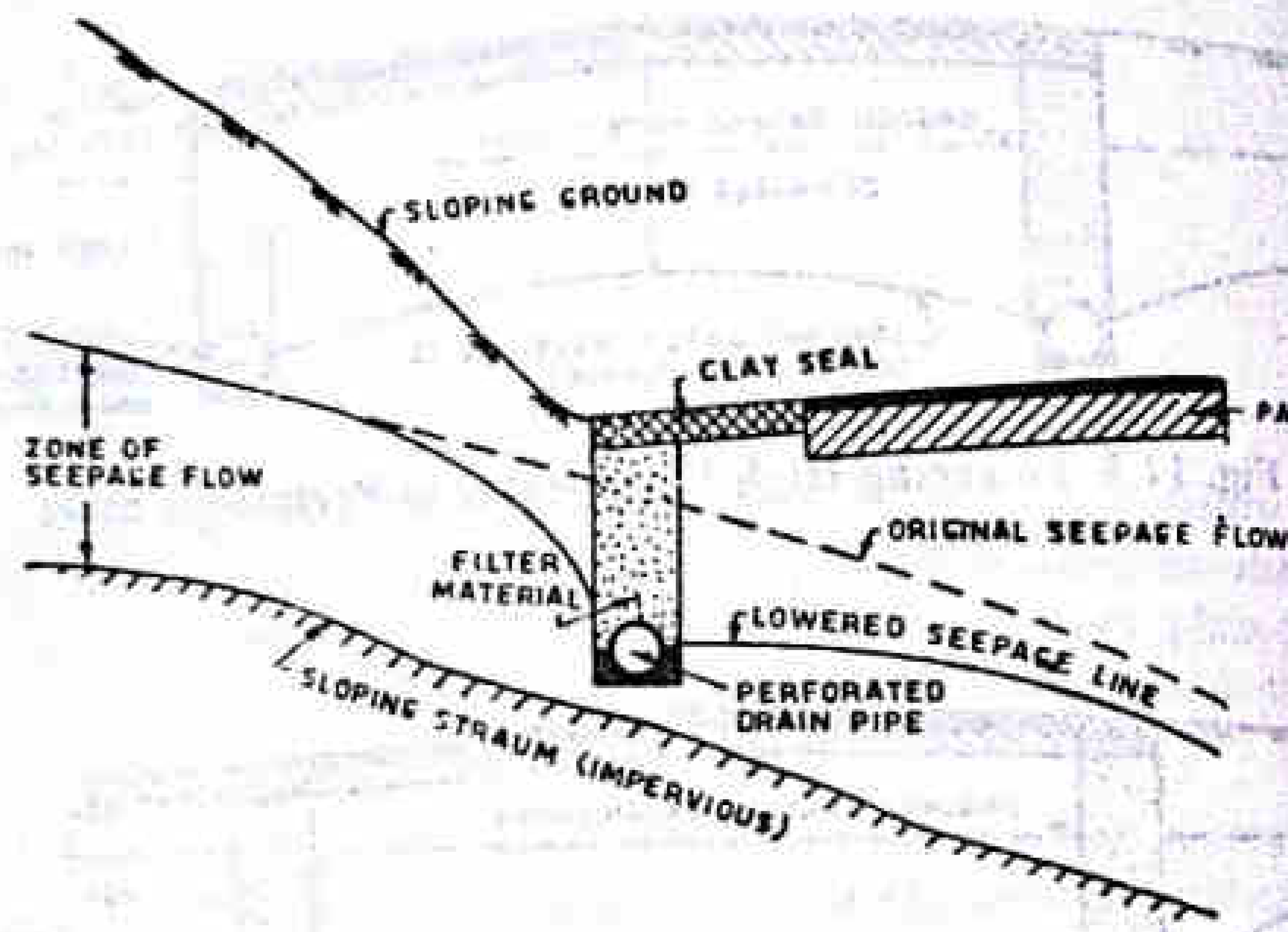

Fig. 11.7 Control of Seepage Flow

11.4.3 Control of Capillary Rise

If the water reaching the subgrade due to capillary rise is likely to be detrimental, it is possible to solve the problem by arresting the *capillary rise* instead of lowering the eater table. The capillary rise may be checked either by a *capillary cut-off* of any one of the following two types :

(i) A layer of granular material of suitable thickness is provided during the construction of embankment, between the subgrade and the highest level of subsurface water table, (See Fig. 11.8). The thickness of the granular capillary cut-off layer should be sufficiently higher than the anticipated capillary rise within the granular layer so that the capillary water can not rise above the cutoff layer.

(ii) Another method of providing capillary cut-off is by inserting an impermeable or a bituminous layer in the place of granular blanket. (See Fig. 11.9).

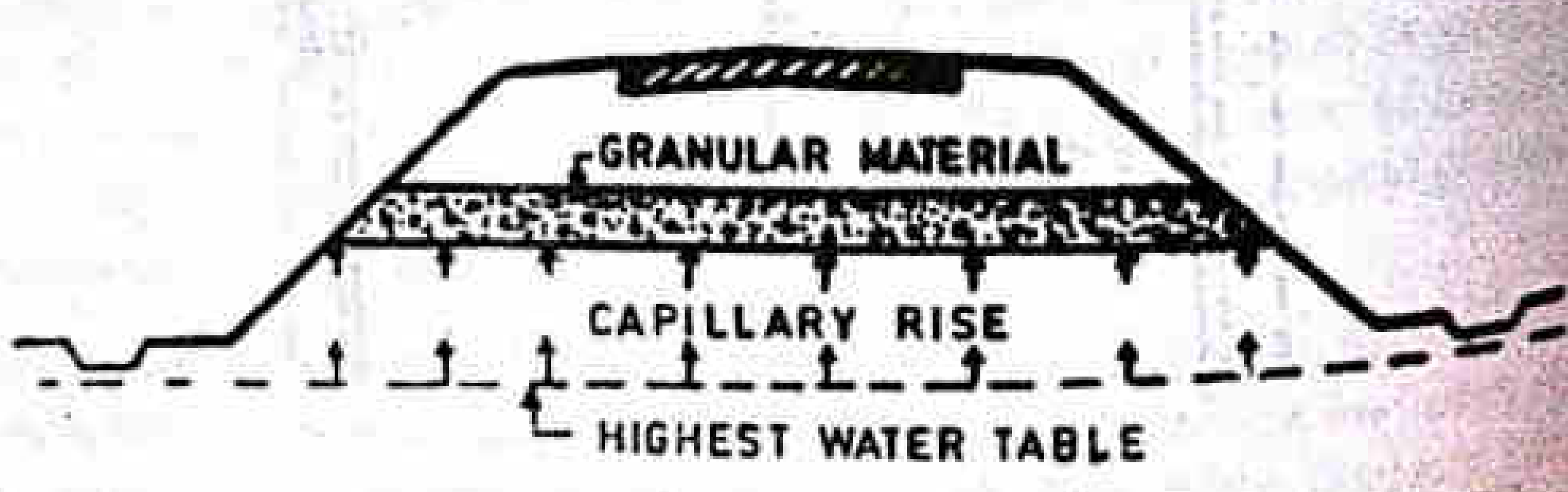

Fig. 11.8 Granular Capillary Cut-off

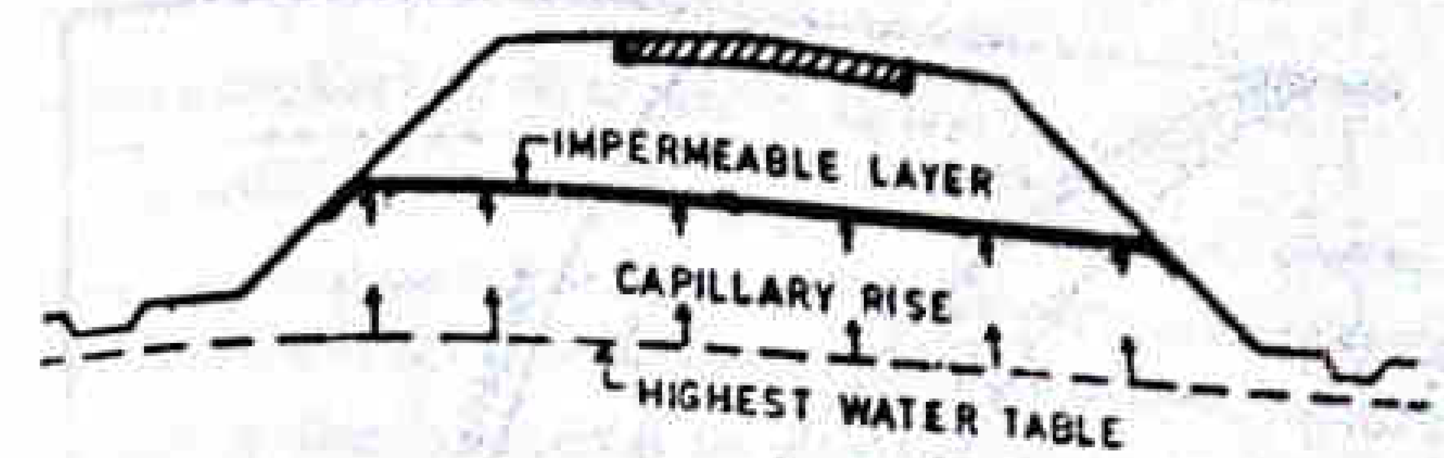

Fig. 11.9 Impermeable Capillary Cut-off

11.4.4 Design of Subsurface Drainage System

The size and spacing of the subsurface drainage system would depend on the quantity of water to be drained off, the type of soil and type of the drains. Mostly this is decided based on experience and other practical considerations. However, proper filter material should be used for back filling the drainage trenches and also for use in all subsurface drainage system.

Design of filter material : The filter material used in subsurface drains should be designed to have sufficient permeability offering negligible resistance to the flow. The filter material should also be designed to resist the flowing of the fine foundation soil resulting in problems like *piping*. Hence the grain size distribution of the filter material is decided based on these two criteria of permeability and piping. The procedure for design of filter is briefly discussed here :

(i) On a grain size distribution chart (percent passing vs particles size on log scale) plot the grain size distribution curve for the foundation soil.

(ii) Find the value of D_{15} size of foundation material and plot a point of particle size $5D_{15}$ of foundation to represent the lower limit of D_{15} size of filter. This is to fulfil the permeability condition given by

$$\frac{D_{15} \text{ of filter}}{D_{15} \text{ of foundation}} \text{ should be} > 5$$

(iii) To fulfil the condition to prevent piping

$$\frac{D_{15} \text{ of filter}}{D_{85} \text{ of foundation}} \text{ should be} < 5$$

Hence plot a point to represent the upper limits of D_{15} size of filter given by 5 D_{85} of foundation.

(iv) Find the size of the perforation in the drain pipe or the gap in the open-jointed pipes and let this be = D_p. Plot a point to represent D_{85} size of filter given by the size 2 D_p.

Refer Fig. 11.10. The shaded area thus obtained represents the region within which the grain size distribution curve of satisfactory filter material should lie.

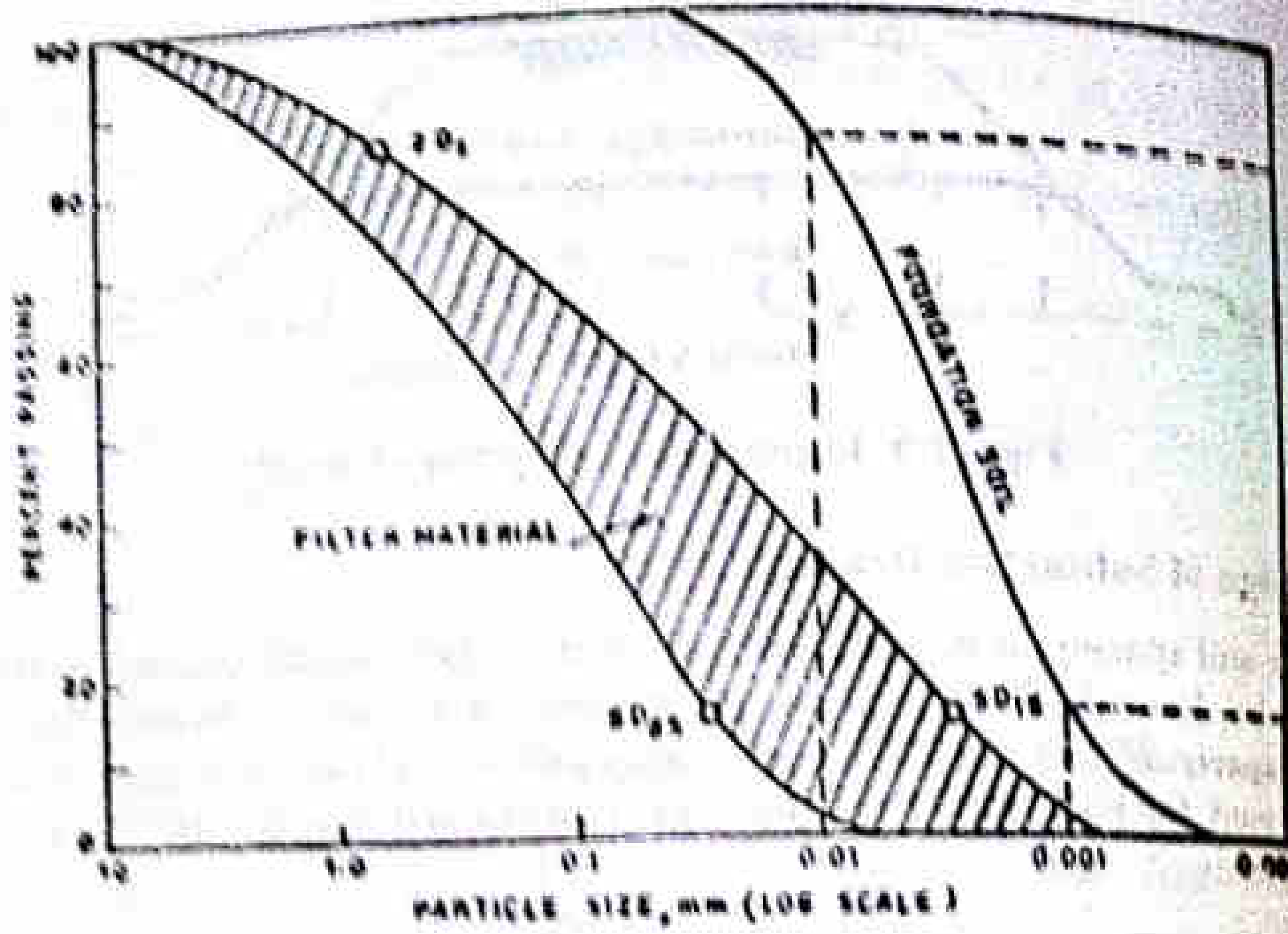

Fig. 11.10 Design of Filter Material

11.5 DRAINAGE OF SLOPES & EROSION CONTROL

Drainage of slopes of embankment, cutting and hill side are of utmost importance to prevent instability of slopes and slides. Soaking of the slope causes increase in stress and reduction in strength. Hence an efficient net work of surface drainage system consisting of intercepting drains and sloping drains to keep the slopes properly drained is very useful for stability. The sloping drains may be provided with lining or pitching or may be filled with gravel. The water from the slopping drains is collected in catch pits and diverted across through the culverts at suitable intervals.

The flow of surface water also frequently causes erosion of soil. This may occur on earth roads if the cross slope is too steep and also on earth shoulders. Surface treatment and stabilization of these surfaces are useful for the control of erosion.

Erosion of soil from the slopes of cut and embankment is detrimental to the slope stability. The erosion depends on several factors such as intensity and duration of rain fall, type and conditions of soil, heights and angle of the slope and climatic conditions. One of the easiest and effective ways of reducing erosion on slope is by *turfing*. Under severe erosion conditions the slopes may be protected by stone pitching riprap or handplaced rocks. Soil stabilization techniques are also useful.

11.6 ROAD CONSTRUCTION IN WATER – LOGGED AREAS

When the subgrade is subjected to soaking condition due to high subsoil water and capillary rise, the area is considered as *waterlogged*. In extreme cases there may be even flooding for prolonged periods. In addition to water-logging if the area is infested with detrimental salts like sulphates, there are additional problems for construction and maintenance of roads in such areas.

When the problem is one of water-logging only without flooding or presence of detrimental salts, the following methods may be adopted :

(i) Depressing the subsurface water level by suitable drainage system.

(ii) Raising the road level by constructing embankment.

(iii) Providing a capillary cut off to arrest the capillary rise of water.

(iv) Providing sufficient pavement thickness in view of the subgrade conditions.

(v) Providing vertical sand drains at suitable spacing and horizontal sand blanket at the top to ensure rapid drainage of water from the foundation soil.

When flooding also is expected for prolonged periods, in addition to the above measures, the road surface may be provided with cement or bituminous concrete pavement. Where traffic is slight, a bituminous surface with seal coat may be sufficient

When detrimental salts are present in the water-logged area, in addition to the above measures which may be adopted with greater care and control, superior pavement construction materials should be used to withstand the detrimental effects of salt. An effective capillary cut-off below the subgrade and a proper water proofing course on the pavement surface and preferably on shoulders also would go a long way in sustaining the detrimental effects of the salts. If the permeability of the foundation soil is low, (as in the case of silty and clayey soils) gravitational drainage would be ineffective. In such cases possibility of using other methods such as electro-osmotic drainage may be considered, if economically feasible.

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PROBLEMS

1. Discuss the importance of Highway Drainage.
2. What are the requirements of a good highway drainage system.
3. Explain how the surface water is collected and disposed off in rural and urban roads. What are the special problems in drainage of surface water in hill roads.

4. Specify the design approach for surface drainage system of a highway.

5. Estimate the design quantity of flow on a road side drain for 25 years period of occurrence of storm from the following data :

   Width of drainage area along bare soil with cross slope 1.0% = 300 m length of drainage area and open drain = 650 m; allowable velocity of flow = 0.5 m/sec; weighted average rolling of runoff cutoff = 0.30

6. The maximum quantity of water to be discharged by the two side drains on a highway section is 1.4 m^3/sec. Design the side drains for the following conditions

   Silty loam, maximum velocity of flow = 0.8 m/sec.

   Roughness coefficient = 0.03.

7. Water flows to a stretch of side drain of length 0.5 km from a total drainage area of 0.35 km^2 consisting of bare earth with average 1% slope, the farthest point from the drainage area to the drain inlet being 0.2 km and also from half the width of two-lane bituminous pavement of length 0.5 km. The run-off coefficient for the bituminous pavement is 0.8 and that of the earth surface is 0.4. Assume 10 year-frequency of occurrence of storm for finding the design quantity of flow into the side drains. Design the cross section and slope of the longitudinal side drain with a bottom width of 0.75 m along clayey soil, if Manning's roughness coefficient is 0.02 and allowable velocity of flow is 1.1 m/sec.

8. Write an explanatory note on cross drainage and drainage structures.

9. Explain with sketches how the subsurface drainage system is provided to lower the water table, and control seepage flow.

10. Indicate how the filter material is designed for use in sub-surface drainage system.

11. Explain the importance of erosion control and drainage of slopes. How are these achieved ?

12. Discuss how the problem of road construction in water logged areas may be solved.

Chapter 12
Hill Roads

12.1 GENERAL CONSIDERATIONS

India has a total area about 259000 sq. km covered with densely forested and thinly populated hills. In the Himalayan region, vegetation and human habitation extends upto high altitudes but habitation is mostly confined to 3000 to 3650 meter above mean sea level.

Due to the mountainous terrain, navigation and rail traffic are not possible. Even air traffic is difficult due to the prohibitive cost of construction of required landing strips. In consideration of strategic need of the country, highest upto elevation of 5500 m are now been negotiated by roads. To provide for communication facilities for the military and to accelerate for development of the regions, the *Border Roads Development Board* was set up by Government of India in the year 1960. The present expenditure on hill roads construction forms about 30 to 40 percent of the total central outlay for roads.

A terrain can be classified into four groups based on the cross slope i.e. the slope approximately perpendicular to the centre line of the highway alignment : (i) plain (ii) rolling (iii) mountainous and (iv) steep terrains.

Hence, a hill road is one which passes through a terrain with a cross slope of 25 percent or more. A hilly or mountainous area is characterised by a highly broken relief with widely differing elevations, steep slopes, deep gorges and a great number of water courses. Owing to complex topography, the route length has to be *ineffectively* increased

Type of terrain	Cross slope, percent
(i) Level (L) or plain	0 to 9.9
(ii) Rolling (R)	10 to 24.9
(iii) Mountainous (M)	25 to 60
(iv) Steep (S)	Above 60

It is the object of the engineer to establish the shortest, most economical and safe route between the obligatory points. This should be done keeping the expenditure of transportation, the wear and tear of the vehicle and the annual maintenance cost to the minimum.

Selection of a suitable alignment in hilly region is a complex job when the valley pattern takes a radial form and where the valley converges into a knot of mountains due to the meeting of several mountains. A thorough knowledge of the geological formations of the area is essential to decide the road alignment, as the process of road construction in mountains disturbs the natural stability conditions. Prevention of soil erosion and stabilization of hill slopes have been major problems in the maintenance of hill roads.

The climatic conditions also influence the progress of hill road construction. In summer, very heavy storms occur in the mountains and about 15 to 20% of the annual rainfall may occur in one storm. The effects of such heavy rainfalls on the construction and maintenance of hill roads are serious. At many locations in the hill roads, the land slides and slips occur blocking traffic during the heavy rainfall. A large number of streams cross the road and hence suitable facility for cross drainage is needed.

Some areas are subjected to heavy winds with a velocity exceeding 100 kmph. On some hill road sections, large quantities of snow are to be cleared during the winter season. National Highway to *Kashmir, Hindustan-Tibbet, Siligure-Gangtok Assam road, Joshimath to Badrinath* are some of the main routes which generally get affected with snow fall and other problems like land slides.

Geometric standards as of plains cannot be adopted in hills, Massive and costly protective works are required at many places in the hill alignment resulting in heavy expenditure. Large quantity of earth work including blasting of hard rock may be needed during hill road construction.

Normally, hill roads are also classified as N.H., S.H., M.D.R., O.D.R. and V.R. as in level terrain.

The Border Roads Organisation (Govt. of India) has classified hill roads as follows :

(i) National Highways

(ii) Class 9 (6 m wide for 3-tonnes vehicles)

(iii) Class 5 (4.9 m wide for 1-tonne vehicles)

(iv) Class 3 (2.45 to 3.65 m wide for jeeps)

12.2 ALIGNMENT OF HILL ROADS

12.2.1 General Consideration

The hill road alignment should link up the obligatory and control points fitting well in the landscape and satisfying the geometric requirements. The best alignment for a hill road is one wherein the total sum of the ascends and descends between extreme points is the least. It is permissible to increase the length as much as 50 times the height saved by a detour.

The various steps necessary in the alignment of a hill road include map study, reconnaissance, trace cut or preliminary survey and detailed survey. It may be advantageous to start the survey from the higher obligatory point. The details of the surveys and general principles of alignment are discussed in articles 3.1 and 3.2. Aerial survey and photogrammetric applications are most useful for the survey of the hills.

Some particulars of special significance are discussed below :

Resisting Length

The resisting length of a road is its effective length taking into consideration the total work done against the resistances. Suppose two stations A and B with difference in elevation 'h' and straight line distance L_0 are to be connected by means of a road. The longitudinal sections of four alternate routes are shown in Fig. 12.1

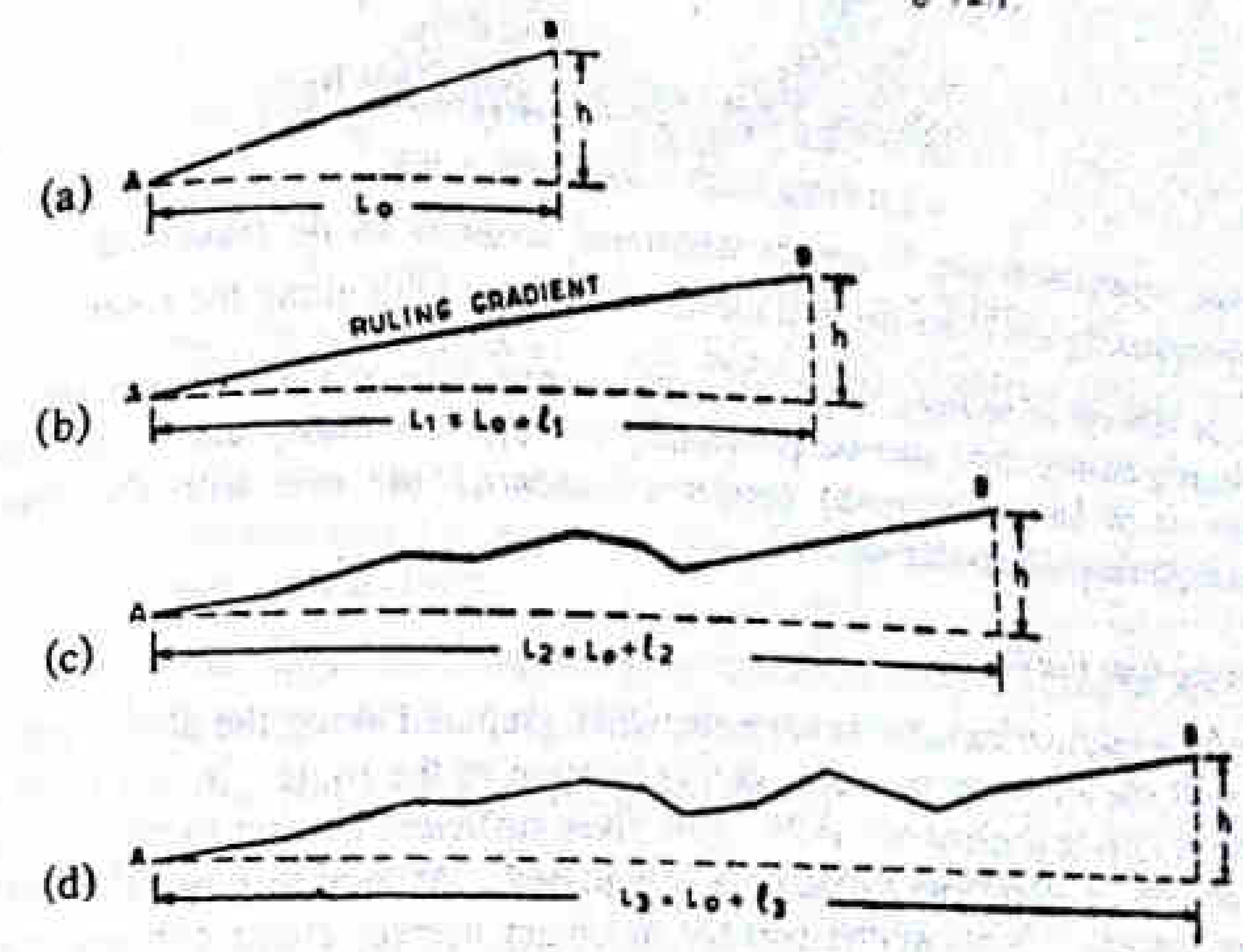


Fig. 12.1 Longitudinal Sections of Alternate Routes

In hill stations the difference in elevation 'h' is likely to be high when compared to the shortest distance L_0 resulting in a gradient steeper than the ruling gradient as in Fig. 12.1 a. Hence it becomes necessary to increase the length at least to $L_1 = (L_0 + l_1)$ so as to have the desired ruling gradient (Fig. 12.1 b).

In actual practice it is not possible to strictly follow such uniform rate of gradient. Figure 12.1c and d illustrate the longitudinal sections of two alternate alignments with lengths $L_2 = (L_0 + l_2)$ and $L_3 = (L_0 + l_3)$ respectively. These two alignments also have ineffective rises and falls, the sum of which are h_2 and h_3 respectively. Here the sum of ineffective rise and fall are obtained by finding the total rise and total fall in excess of *floating gradient* then subtracting the actual difference in elevation 'h' between the two stations A and B. Ineffective rise and fall cannot always be avoided in such alternate routes due to the topographic conditions enroute.

The resisting lengths of the different routes are compared by considering amount of work required to move a load over these routes.

The total work done in case (a) (Fig. 12.1a) in moving a load W from station A to B along the shortest length L_0 up to height, h

$$= W f L_0 + W h = W f (L_0 + h/f) \quad (12.1)$$

$$= W f L_r$$

Here

L_r = resisting length = $L_0 + h/f$

f = coefficient of frictional resistance.

In case (b) the resisting length

$$L_{r1} = L_1 + h/f \quad (12.2)$$

Similarly in alignments (c) and (d) the values of resisting lengths are given by

$$L_{r2} = L_2 + \frac{h+h_2}{f} = L_o + l_2 + \frac{h+h_2}{f} \quad (12.3)$$

and

$$L_{r3} = L_3 + \frac{h+h_3}{f} = L_3 + l_3 + \frac{h+h_3}{f} \quad (12.4)$$

In these equations l_2 and l_3 are the additional distance to be traversed along the alternative routes, h_2 and h_3 are the total uneffective rise and fall along the routes.

Thus the concept of resisting length gives the correct idea of various alternate routes while selecting an alignment after the preliminary survey. Obviously out of the alternate alignments which fulfill the desired geometric standards, the one with the minimum resisting length should be preferred.

Trace Cut for Hair Pin Bends

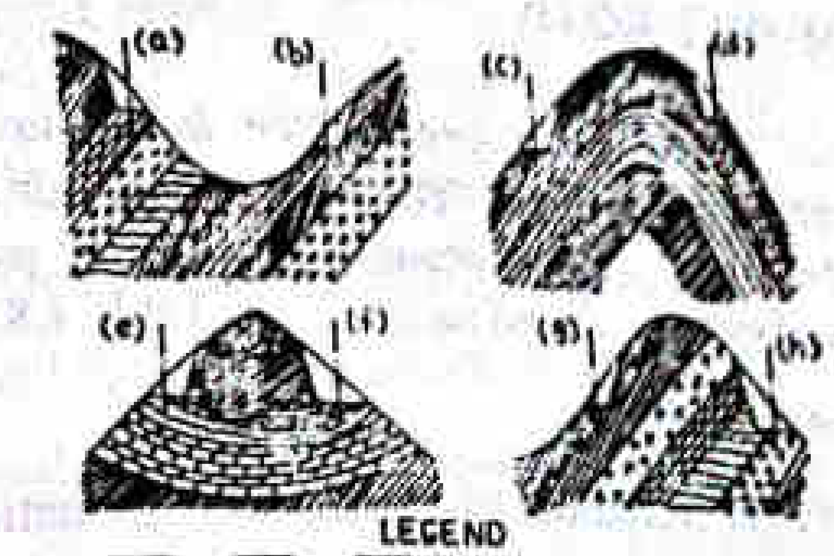
Trace cut is a narrow track, 0.6 to 1.0 metre wide, prepared along the alignment of the hill road to enable access for inspection during location of the route. In hill roads trace cut is done at a ruling gradient of 1 in 25. This gives sufficient margin to obtain a ruling gradient of 1 in 20 when final formation is completed. At *zigs* this practice leads to difficulties, because it is not always possible to obtain correct curve compensation to gradient. The ruling gradient if changed to almost level at the *hair-pin bends* or sharp curves to ensure that finished gradient is correct. Hence detailed survey is necessary at each sharp curve, particularly at hair-pin bends.

Sharp curves of radius below 18 meter often occur either in re-entrants or in thin spurs. A re-entrant curve is provided to negotiate a deep but narrow valley, forming as open bend. Part of the re-entrant curve may be filled up for providing desired width at the bend. Sharp curves while going round a thin spur of hill forming a blind bend, would necessitate cutting-in, so as to provide sufficient visibility and width of formation.

Geological Considerations

If the geological and hydrological conditions are not taken into account, the construction of the road may disturb the natural slope stability. Degree of stability of slope depends on type of rock, inclination or *dip* of stata and presence of ground water. Dip of the strata should be as small as possible or alternatively, be inclined away from it. See Fig. 12.2. The sides a, e, f and h are stable locations, whereas b, c, d and g are unstable locations.

Stratified sedimentary rock often occurs as *folds* which may be concave (*syncline*) or convex (*anticline*). The inclination of the folds may vary from horizontal to vertical. The folds may have various *faults* too. The inclination of strata varies considerably from place to place and foldings, inversions and faults exist along the line of movement. These determine the physical characteristics of the formation. Roads are generally taken in cutting along hill face. The stability of the face is an important feature for the safety of the road. Careful observations give some idea of the geology of the region but purely superficial observation lead to serious errors.



1. Sandy loam 2. Clay and clay loam 3. Clay shale
4. Granite 5. Lime stone 6. Rubble

Fig. 12.2 Stability of Roads Based on Inclination of Strata

The side hazard can be checked by adjusting the alignment so that the cut slopes intercept the beds at a more favourable angle to the bedding planes. The alignment should be so adjusted that the bedding planes of the rock tends to *dip* away from the cut slopes rather than towards them.

In locating an alignment, consideration should be given to some of the additional situations which may cause land slides due to proposed cuts or fills. These are :

(a) Restriction of ground water flow by the side fill

(b) Over loading of relatively weak underlying soil layer by fill

(c) Over loading of bedding planes by heavy side fill

(d) Over steepening of cuts, unstable rock or fill

12.2.2 Alignment Survey

The alignment of hill road is fixed in the three stages :

(i) Reconnaissance

(ii) Trace cut and

(iii) Detailed surveys

The general route for the alignment is selected during the reconnaissance. First the topographical geological and meteorological maps of the area and aerial photographs (if available) are studied. This may be followed by aerial reconnaissance where it is necessary and feasible. Subsequently the ground reconnaissance and detailed study of difficult stretches are carried out.

The route selected during the reconnaissance is translated on the ground during the trace cut so as to provide an access for subsequent detailed surveys. 1.0 to 1.2 m wide track is constructed, with easier gradients than the specified gradients. Instead of cutting into hard rock, access is achieved by means of dry rubble filling or walls.

During the detailed surveys, bench marks are fixed and the longitudinal and cross sections are obtained. A strip covering about 15 m on either side on straights and 30 m at sharp curves may be surveyed. Necessary adjustments are made in the alignment to suit the design of horizontal curves and hair pin bends. The centre line is marked by stakes and reference pillars. Hydrological and soil investigations are carried out for the route.

12.3 GEOMETRICS OF HILL ROAD

The geometric standards for gradient, superelevation, and radius of curves etc. on hill roads are different from those in the plains. The main reasons for the difference are the topography and other problems in alignment of hill road as already explained. Consequently the speed standards are also to be low. (See Table 4.8).

Width of Pavement, Formation and Land

The recommended widths of pavement or carriageway, formation and land for hill roads in India are given in Table 12.1.

Table 12.1 Width of Pavement, Formation and Land

Highway classification	Pavement width, m	Roadway width, m (excluding side drains and parapets)	Right-of-way width, m	
			Normal	Exceptional
NH & SH				
Two-lane	7.00	8.80	24	18
Single-lane	3.75	6.25		
MDR	3.75	4.75	18	15
ODR	3.75	4.75	15	12
VR	3.00	4.00	9	9

At stretch on hard rock, the shoulders may be reduced by 0.4 m on either side on two-lane roads and by 0.2 m in other cases. The minimum set back for building line beyond the right-of-way should be 5 m in normal cases and 3 m in exceptional circumstances.

Camber or Cross-Fall

Steeper cross slope or camber is adopted for hill roads and recommended values are given in Table 12.2.

When the road has longitudinal gradients greater than 1 in 20, flatter camber may be provided.

Table 12.2 Recommended values of camber

Type of Surface	Camber, percent
Subgrades, earth roads and shoulder	3.0 to 4.0
Gravel and W.B.M. surface	2.5 to 3.0
Bituminous surfacing	2.5
High type bituminous surface & CC	2.0

Sight Distance

The stopping sight distance is calculated from the relation :

$$SSD = 0.278\,Vt + \frac{V^2}{254\,f} \qquad (12.5)$$

Where V = design speed, kmph

t = reaction time, taken as 3 seconds

f = coefficient of friction, assumed as 0.4

Safe stopping sight distance for various speeds given by IRC are given below :

Speed, kmph	20	25	30	40	50
SSD, m	20	30	35	50	70

The overtaking sight distance is calculated from the relation :

$$OSD = 0.556\,V_b + 2\,s + 0.278\,TV_b + 0.278\,VT$$

where V = speed of overtaking vehicle, kmph

V_b = speed of overtaken Vehicle = $(V - 16)$ kmph

s = spacing of moving vehicles = $(0.2\,V_b + 6)$ m

T = overtaking time = $\sqrt{\frac{14.4}{A}}$, secs.

A = acceleration in kmph/sec. taken as 4.72, 4.45 and 4.0 for speeds of 30, 40 and 50 kmph respectively.

Minimum overtaking sight distance specified are :

Speed, kmph	30	40	50
OSD, m	90	145	210

Superelevation

The superelevation to be provided at horizontal curves of hill roads is calculated from the formula :

$$e = \frac{V^2}{225\,R} \qquad (12.6)$$

IRC specifies that the superelevation should not exceed 7 percent in sections of hill roads which get snow bound and 10 percent in other places.

Radius of Horizontal Curve

The minimum radius of horizontal curves in hill roads, R (min) is calculated from the formula :

$$R\,(min) = \frac{0.008\,V^2}{e + f} \qquad (12.7)$$

The lateral friction factor f is taken as 0.15. The minimum radii recommended for various classes of hill roads are given in Table 12.3.

Table 12.3 Minimum Radii of Curves in Hill Roads

Category of Roads	Minimum radius, metre			
	Mountainous terrain		Steep terrain	
	not snow bound	snow bound	not snow bound	snow bound
N.H. & S.H.	50	60	30	33
M.D.R.	30	33	14	15
O.D.R.	20	23	14	15
V.R.	14	15	14	15

Reverse curves are designed to have a minimum radius of 30 m for the compound curves and a straight distance of 9 m between their transitional ends. In exceptional cases, the radius can be reduced to 22.5 m and the straight distance is dispensed with.

Widening at Curves

Extra width of carriageway W_e at horizontal curves is calculated from the relation :

$$W_e = \frac{18n}{R} + \frac{0.1V}{\sqrt{R}} \tag{12.8}$$

where n is the number of lanes.

The recommended values of extra widening on single and two-lane pavements at curves are given below for various speeds.

Radius of curve, m	14-20	20-30	30-60	60-150	above 150
Extra width of *single-lane* roads, m	1.5	1.2	0.9	0.6	Nil
Radius of curve, m	30-40	40-60	60-100	100-150	above 150
Extra width of *two-lane* roads, m	1.5	1.2	0.9	0.6	Nil

The formation or roadway width also should be increased at the horizontal curves. On single-lane National and State highways the extra width of the roadway is taken equal to the extra width of carriage way itself. In all other cases the roadway width is suitably increased so as to provide a shoulder width of atleast 0.5 m on either side after the carriageway has been widened.

Setback Distance

As it is not practicable to provide visibility corresponding to overtaking sight distance all along the hill road, the alignment is made so as to provide atleast the safe stopping sight distance. Accordingly the minimum set back distance on the inner side of horizontal curves for various speeds and radii of curves have been specified by the Indian Roads Congress.

Transition Curves

The length of transition curves is to be calculated from the formula :

$$L_s = \frac{0.0215\,V^4}{CR} \tag{12.9}$$

Here $C = \frac{80}{V + 75}$ (maximum values of 0.76 for speeds less than 30 kmph)

L_s = length of transition, metre

R = radius, metre

V = design speed, kmph

The minimum lengths of transition recommended by I.R.C. are 10 m for design speed up to 40 kmph and 20 m for design speed 40 to 50 kmph.

Gradients

The ruling and limiting gradients in mountainous terrain and in steep terrain over 3000 m height above mean sea level are 5 and 6 percent respectively. The ruling and limiting gradients in steep terrain upto 3000 m height above MSL are 6 and 7 percent. At high altitudes (> 3000 m) as the pulling power of engines decrease due to reduction in oxygen supply, the design values of steepest gradients should be lower. Exceptional gradients steeper than the limiting gradient may be sparingly used, separated by a minimum length of 100 m. At horizontal curves, the percentage compensation in gradient may be provided using the formula $\frac{30 + R}{R}$ with a maximum of $\frac{75}{R}$. The compensated gradients may not however be flatter than 4 percent.

Summit and Valley Curves

The length of summit and valley curves may be calculated as explained in Art. 4.5.3.

Hair Pin Bends

Because of precipitous rock, deep valley, steep ascends to obligatory points and presence of innumerable gorges, hair pin bends are unavoidable on hill roads. Within limits of the available turning angle, it is often very difficult and sometimes even impossible to lay out curves following normal geometric standards of design. A hair pin bend is located on a hill side having the minimum slope and maximum stability. It must also be safe from view point of land slides and ground water.

Hair pin bends with long arms and farther spacing should be preferred. This will reduce construction problems and expensive protective works. The following design criteria are adopted for planning hair pin bends :

(a) Straight length between two successive hair pin bends should be minimum of 60 m excluding the length of circular and transition curves. This length further depends upon the hill slopes to avoid costly protective measures between the upper and lower arms of the bends.

(b) Minimum design speed = 20 kmph

(c) Minimum radius of the inner curve = 14 m

(d) Minimum length of transition = 15 m

(e) Superelevation in circular portion of the curve = 1 in 10

(f) Minimum width of carriage way at the apex of the curve are respectively 11.5 and 9.0 m for two-lane and single-lane pavements of National and State Highway. The minimum width for MDR and ODR is 7.5 m.

(g) The maximum and minimum gradients are 1 in 40 and 1 in 200 respectively at the curve.

(h) Approach gradients should not be steeper than 5 percent for 40 metre.

(i) For good visibility at the hair-pin bend, the island portion shall be cleared of all the trees etc.

Figure 12.3 shows a hair pin bend and few typical cross sections.

Cut slopes

The following side slopes are recommended at cuts depending upon the soil types :

Type of soil	Side slopes (vertical to horizontal)
Ordinary soils including moorum and hard clay	1 : 1 to 1 : 1½
Disintegrated rock or conglomerate	1 : 1/2 to 1 : 1/4
Soft rock and shale	1 : 1/4 to 1 : 1/8
Medium rock	1 : 1/12 to 1 : 1/16
Hard rock	Nearly vertical (or half-tunneling if the height of cut exceeds 7.5 m)

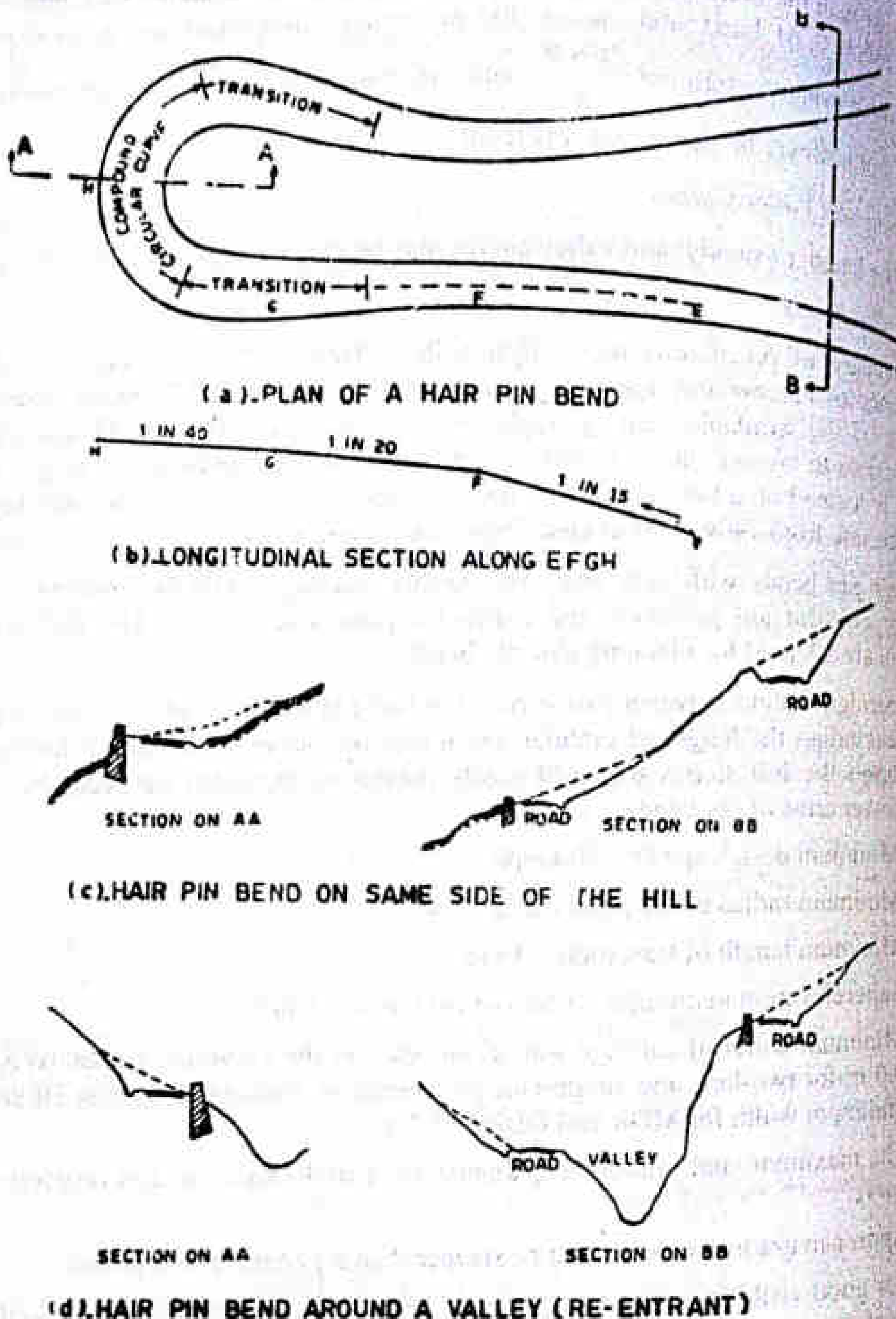


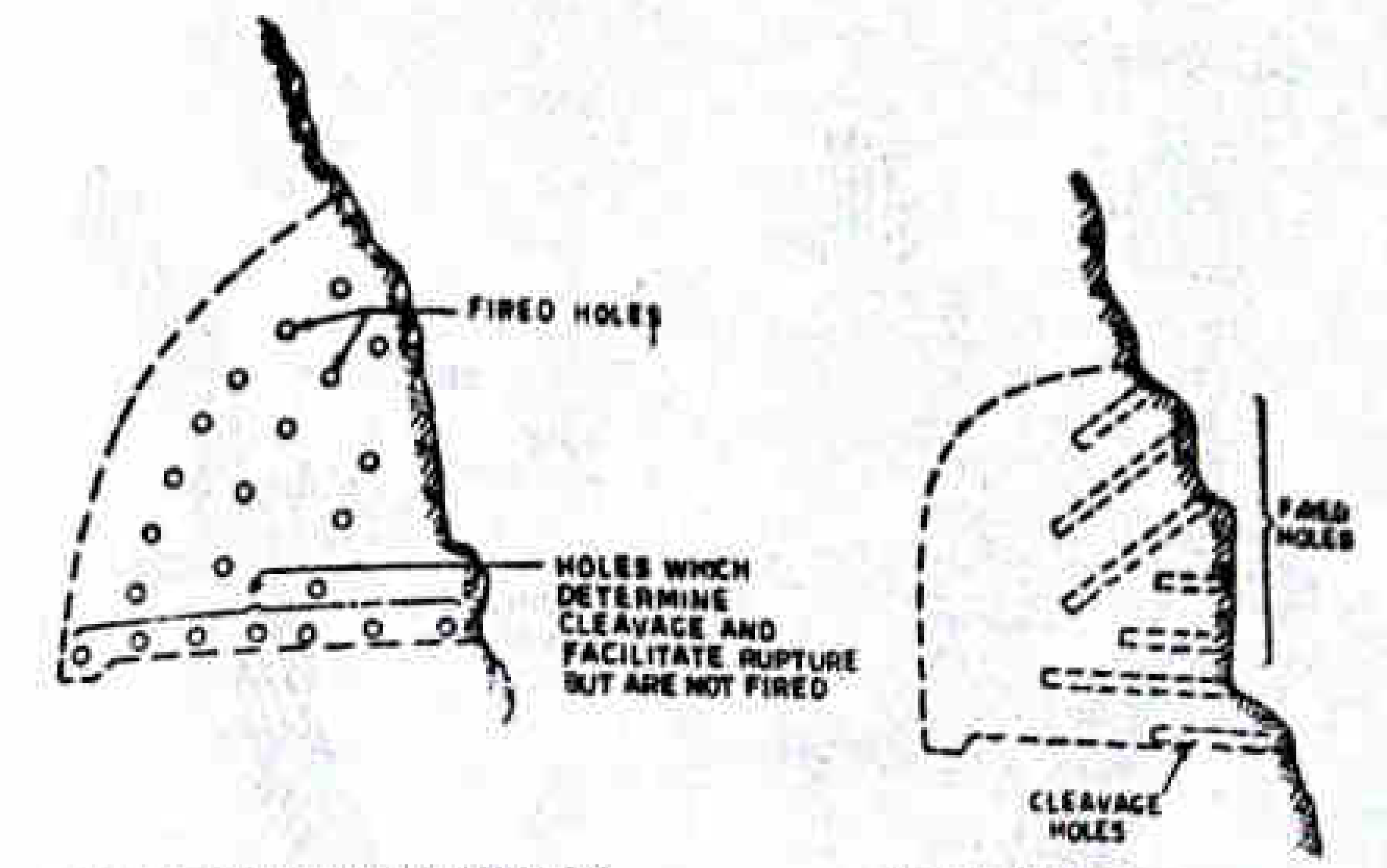
Fig. 12.3 Hair Pin Bends

However detailed investigations are needed in deep cut and where problems of instability are likely.

12.4 DESIGN AND CONSTRUCTION OF HILL ROADS

Rock Cutting

If the rock strata slopes downward into the hill-side, the rock is permitted to overhang the road forming a half tunnel. Blasting is done either from face or from one or both sides. See Fig. 12.4. If the strata is inclined towards the hill slope, cutting is continued



BORE HOLES FOR END ATTACK BORE HOLES FOR FACE ATTACK

Fig. 12.4 Rock Blasting

until the inner slope is at a safe angle to prevent slipping. In such a case, blasting and cutting is commenced from top. If the rock is soft or loosely stratified, the cutting is commenced at point on the hill-side far above the formation level for obtaining sufficient batter for the inner slope. Blowing out the cliff face by firing large mines and then dressing the slope, is another alternative. In actual construction, there is great difficulty in removing rock exactly to the required levels and grade. Unless special care is taken, blasting may be slightly short of requirement or it can be over-blasting. During blasting the drain on hill-side is also formed. Blasted stones are used for retaining walls, back-filling high retaining walls and aprons etc.

Precipice Work

Where the time available does not allow for blasting and tunnel work, *cliff galleries* and *cradles* are resorted for the negotiation of cliffs and precipices. These are suitable only for light vehicles or foot traffic and considered only for short term use and not as a permanent road way for regular traffic. See Fig. 12.5 and 12.6. It is important that the strata should dip inwards from the face in order to ensure safe attachment for the jumpers and holdfasts and to lessen the risk of rock falls. If the top of the precipice is inaccessible, it is necessary either to start from below or to work forward from the farthest point of the road. A log or light gangway is projected forward beyond the end to provide a platform from which the holes are drilled and the supports are fixed in advance.

When boring is not practicable, a light roadway may be carried on timber cradles suspended by means of wire cables from jumpers driven at the top of the cliff, as shown in Fig. 12.6. The bearer of the cradle should tilt upwards; otherwise when the cradle draws away from the face of the cliff; the surface will slope outwards and become dangerous.

Retaining Walls

Retaining walls are most important structure in hill road construction to provide adequate stability to the roadway and to the slope. Retaining walls are constructed on the valley side of the roadway and also on the cut hill side to prevent land slide towards the roadway.

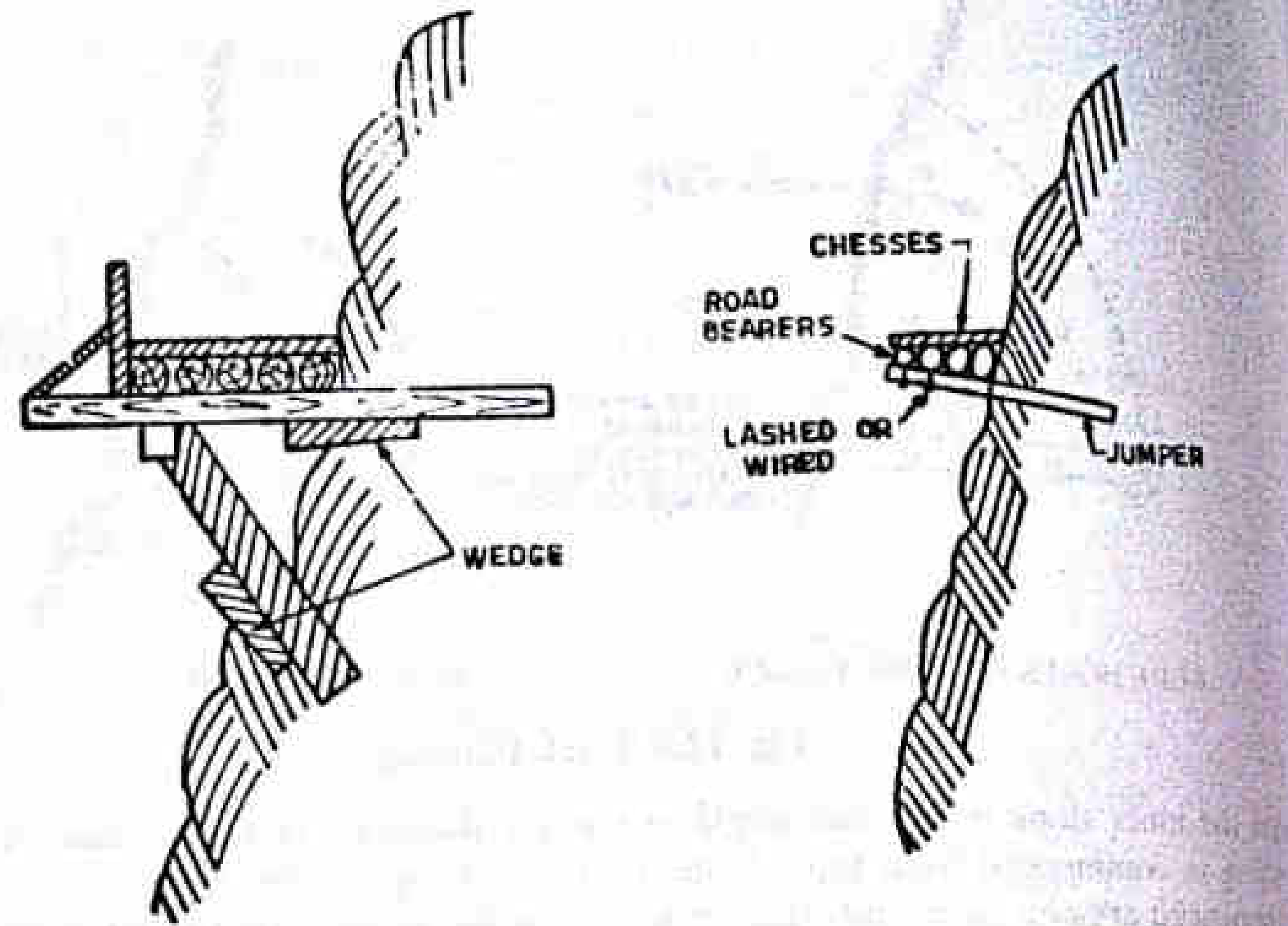


Fig. 12.5 Cliff Galleries

Dry stone masonry is preferred to masonry in mortar as the former permits easy drainage of seeping water. The design of retaining walls is based on the thumb rules and performance of existing structures in similar regions. *Harger* and *Bonney* suggest a section of 0.5 H with a minimum width of 0.45 m to 0.60 m at top and a front batter of 1 in 4 with the rear side vertical. A typical design is given in Fig. 12.7.

For heights greater than 6 m, sections of (0.4 H + 0.3) to (0.5 H + 0.6) m are adopted with a top width of 0.75 m. Also bands of coursed rubble masonry in cement mortar at vertical and horizontal intervals of about 3 m each are constructed as shown in Fig. 12.8, for added stability. Here the rear face is vertical and filled with boulders and stones to improve drainage and to resist earth pressure for half the height. The front batter works out to 1 : 2½ to 1 : 3.

Revetment Walls

The embankment slopes are normally protected with rough stone pitching about 30 cm thick in order to avoid erosion due to flow of water. See Fig. 12.9. If the stoping length is too long, it is preferable to construct a toe wall as shown in Fig. 12.10 to support the embankment and depending upon the slope available. Where the cutting slope is steep and contains loose or scourable soils, slips are likely to occur. In such locations, revetment walls of dry stone masonry are constructed to retain the soil on the cutting side to prevent occurrence of any such slips.

Revetment walls of heights 13.5 m and 28.5 m have been constructed in the *Bodimettu Ghat* road in *Tamil Nadu*.

Foundations of Retaining Walls

In hilly regions, the presence of hard gravelly soil or even soft rock may be sometimes misleading as these may occur in thin section over lying sloping rocks. So, whenever the

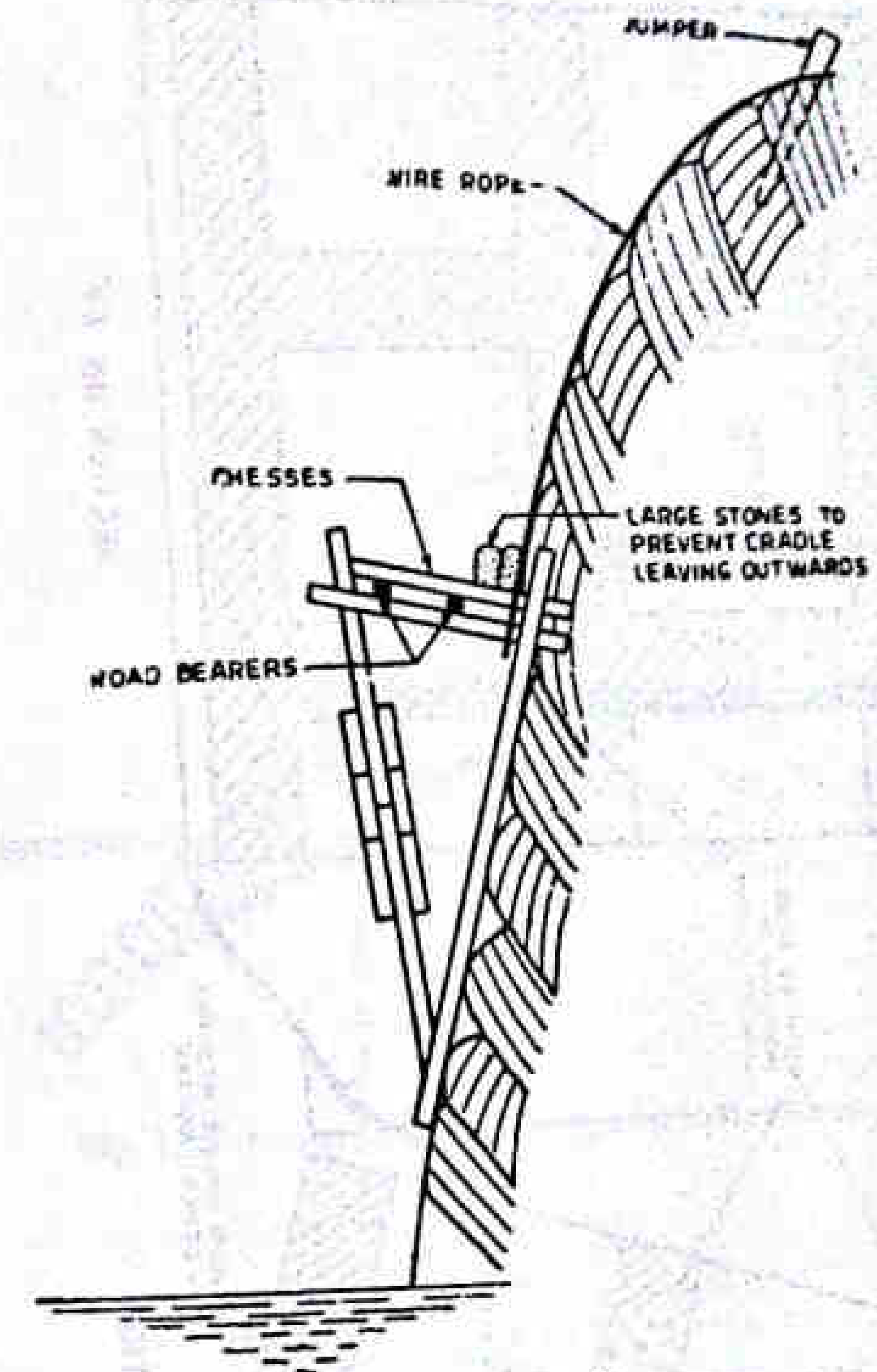


Fig. 12.6 Cradle

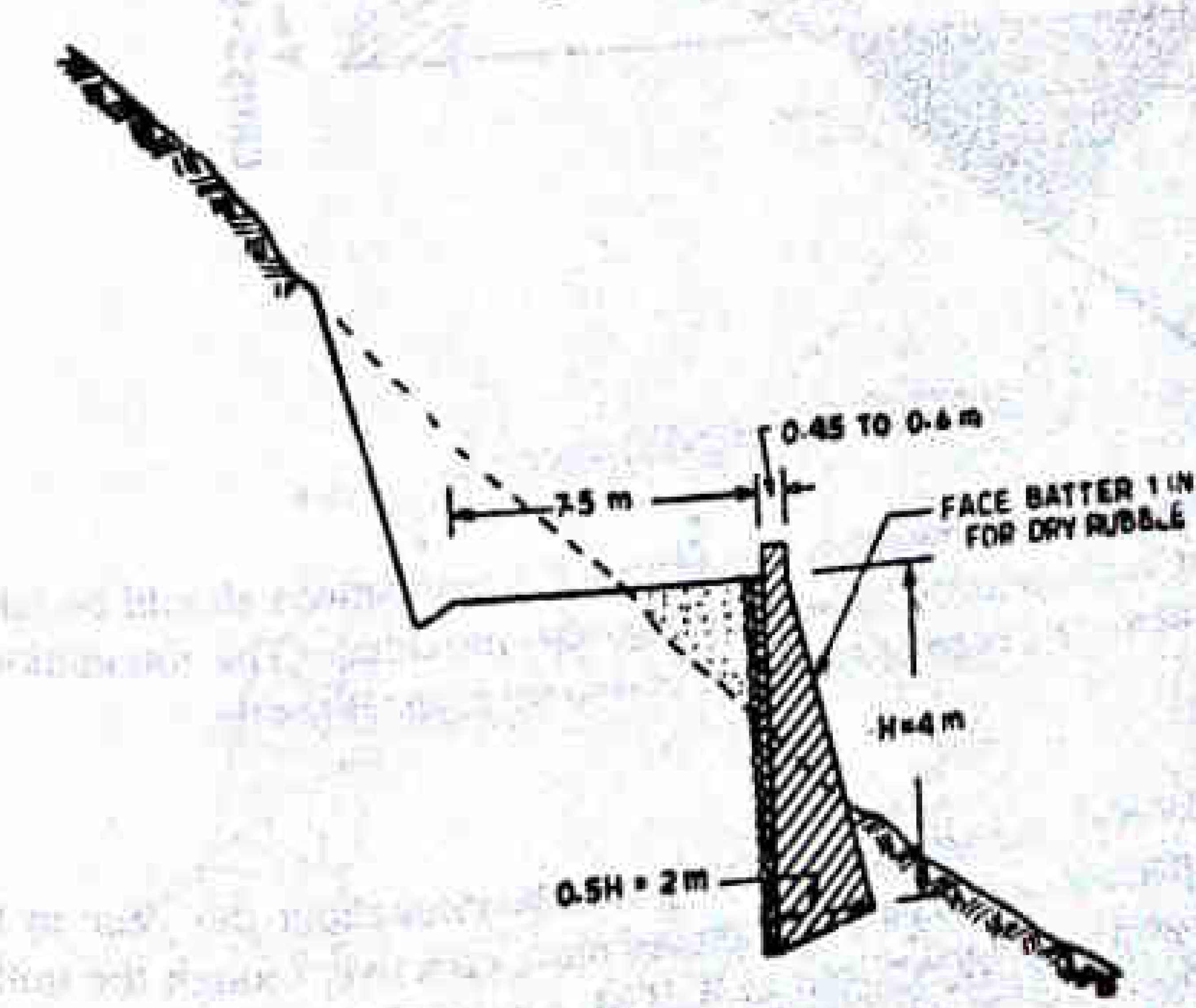


Fig. 12.7 Typical Retaining Wall

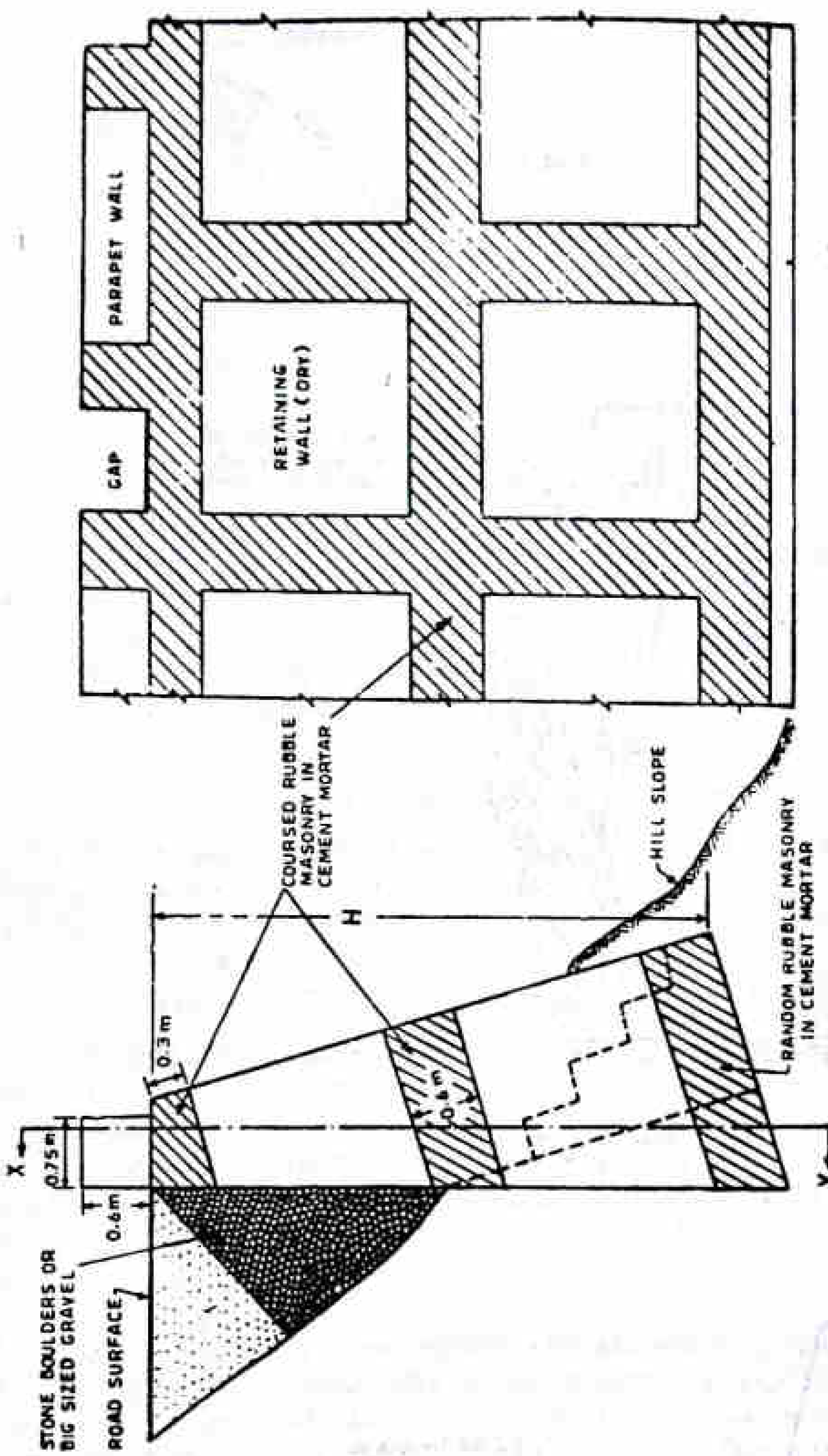


Fig. 12.8 High Retaining Wall

presence of hard rock at shallow depth is indicated, the foundations should be taken upto the rock bed. In addition to this, dowel bars are also provided. The foundation bed is provided with a downward slope of 1 in 6 or 1 in 4 towards the hill side.

Pavement Type

Because of the high intensity of rain fall generally throughout the year in the hill regions, an impermeable type of pavement proves more effective, though the initial cost may be high. A permeable surface such as W.B.M. gets eroded by the heavy rains and regular maintenance cost comes out to be high.

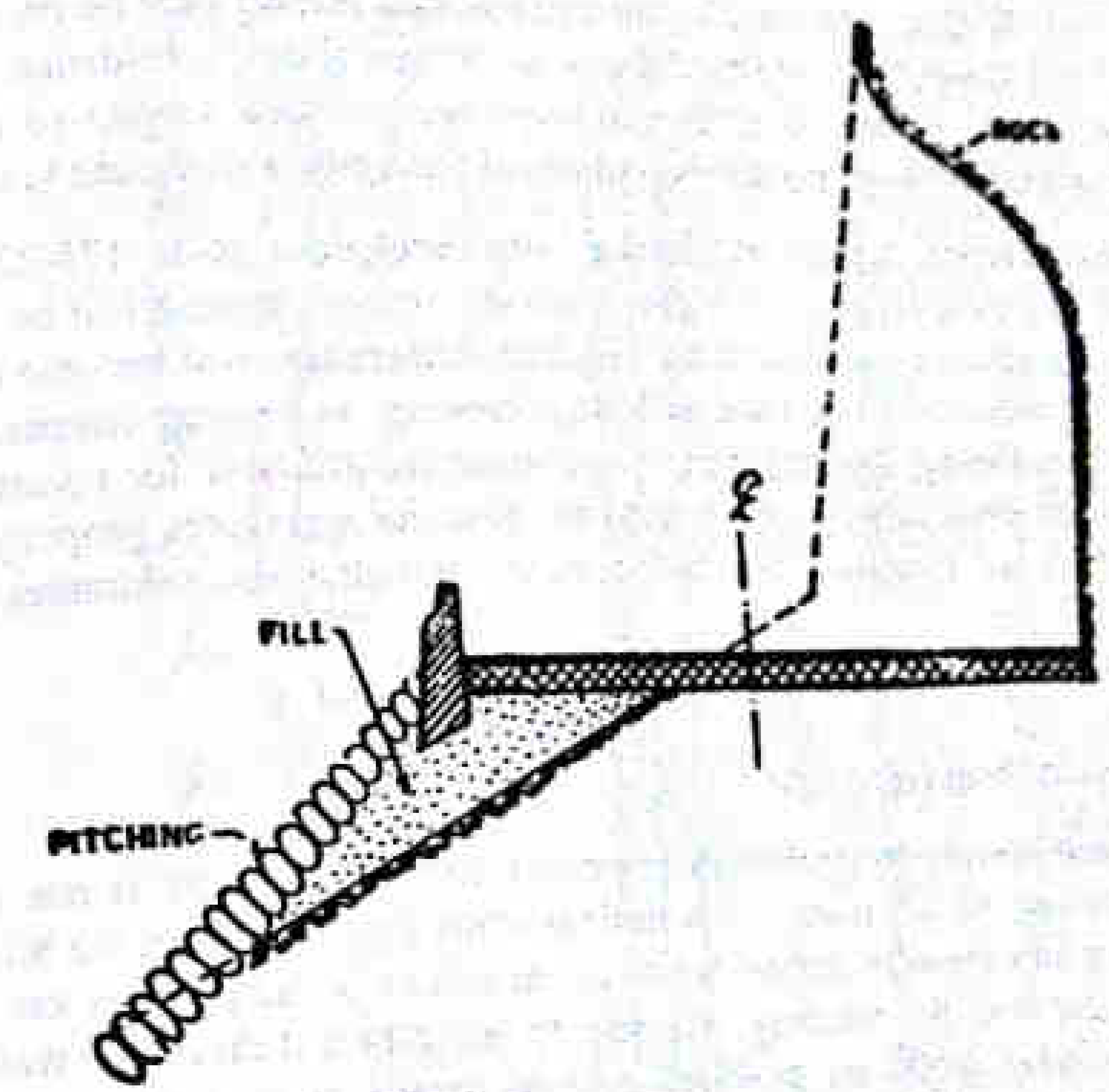


Fig. 12.9 Protection of Hill-slope by Stone Pitching

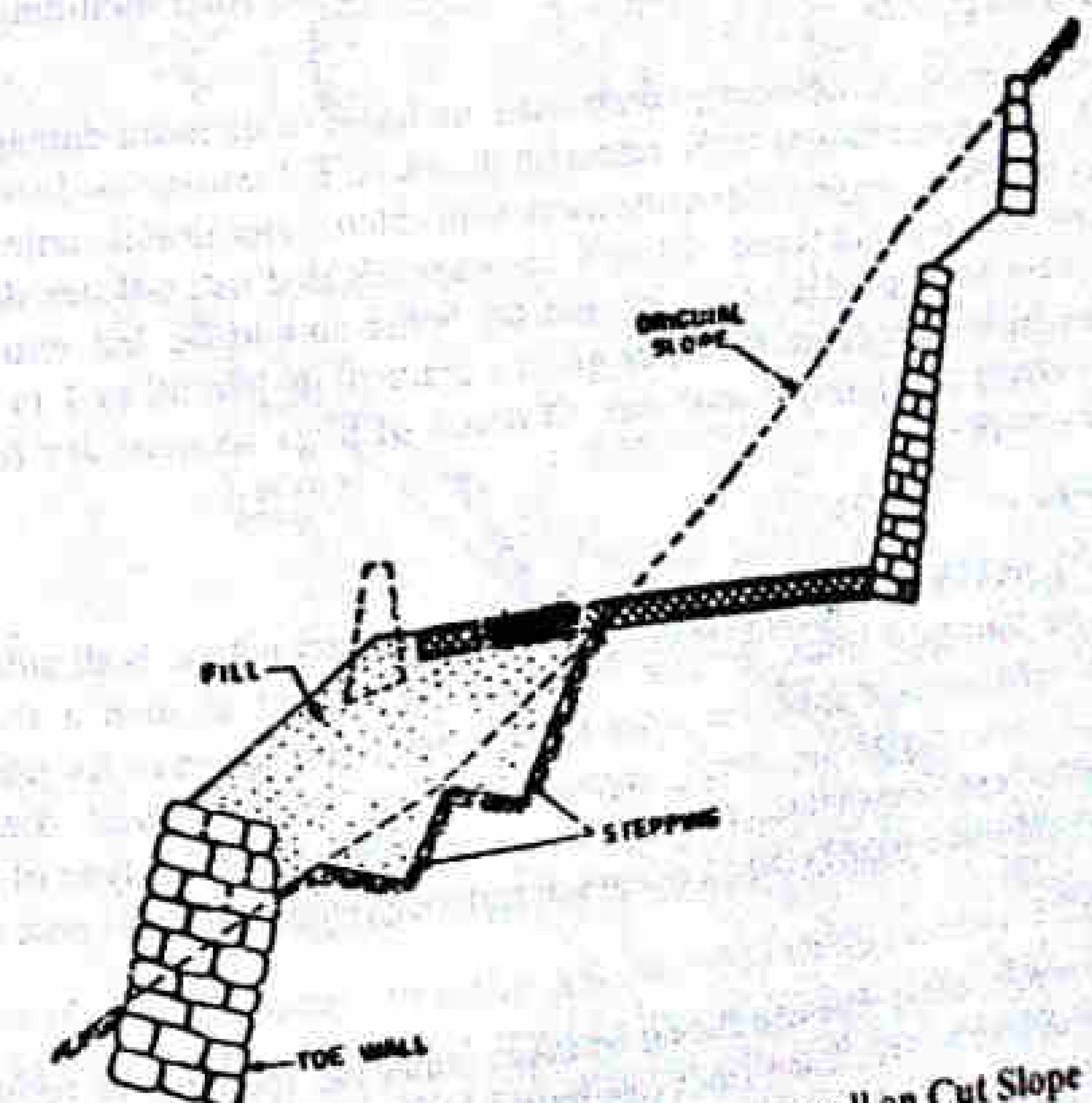


Fig. 12.10 Toe Wall with Revetment wall on Cut Slope

The bituminous pavements are therefore preferred on hill road. Cement concrete pavements are not considered suitable because of its high initial cost and delay in construction. Since frequent damages are expected in hilly areas, a flexible pavement can be more easily and cheaply repaired than a cement concrete pavement. A flexible type of pavement with W.B.M. base course and bituminous surfacing may be recommended for hill roads. But in localities where damages due to land slides, subsidence, snow fall etc. are not anticipated, cement concrete pavement are provided subject to availability of funds. The atmospheric temperature variations of the region also should however be low.

I.R.C. recommends the use of bitumen with penetration grade 175/225 for ground macadam in very cold regions. A higher viscosity bitumen, though can be sprayed to or mixed with aggregates by heating at the required temperature, will become brittle in very cold regions. Cutback are also used with slight heating. In freezing temperature cutback grade RC-3 is preferred over bitumen if equipment are available for construction, since bitumen gets hardened rapidly and it does not coat the aggregates properly. In border areas where there will be mostly tank traffic, mastic asphalt is also recommended.

12.5 DRAINAGE IN HILL ROADS

Drainage of Water from Hill Slope

Surface water flowing from the hill slope towards the road way is one of the main problems in drainage of hill roads. It is desirable that the water from the hill side is not allowed to flow into the side drains due to the problems in maintaining the side drains intended for water from the roadway. In order to intercept and divert the water from the hill slope *catch water drains* are provided, running parallel to the roadway. Water from the catch water drains is diverted by sloping drains and carried across the road by means of culverts.

Figure 12.11 shows the layout of drainage system in hill road including catch-water drains and sloping drains.

Catch-water drains, if improperly constructed are liable to do more damage than good to the road. Recommendations in the report on the ECAFE *Seminar on Low Cost Roads* held in January, 1958, suggest dispensing away with catch-water drains, unless the drains are lined and properly maintained. ECAFE recommends that catch-water drains should be located from the top edge of cutting and the water should be led into the nearby culvert. The catch water drains should be given a gradient of 1 in 50 to 1 in 33 to avoid high water velocity and possible wash out. If drains of large sections are required, the bed and sides are paved.

Road-side Drains

Side drain is provided only on the hill side of the roads and not on both sides. Due to limitation in the formation width, the side drains are constructed to such a shape that at emergency the vehicles could utilize this space for crossing at low speed or for parking. The usual types of side drains are *angle, saucer* and *kerb and channel* drains. See Fig. 12.12. Ministry of Transport recommends the use of kerb and channel type of side drain. In hard rock the gutters need not be paved; but in soft soil rough stone paving is necessary.

Cross Drainage

As far as possible, cross drainage should be taken under the road and at right angle to it. At the head of small cross drains catch pits must be provided to collect the stones and

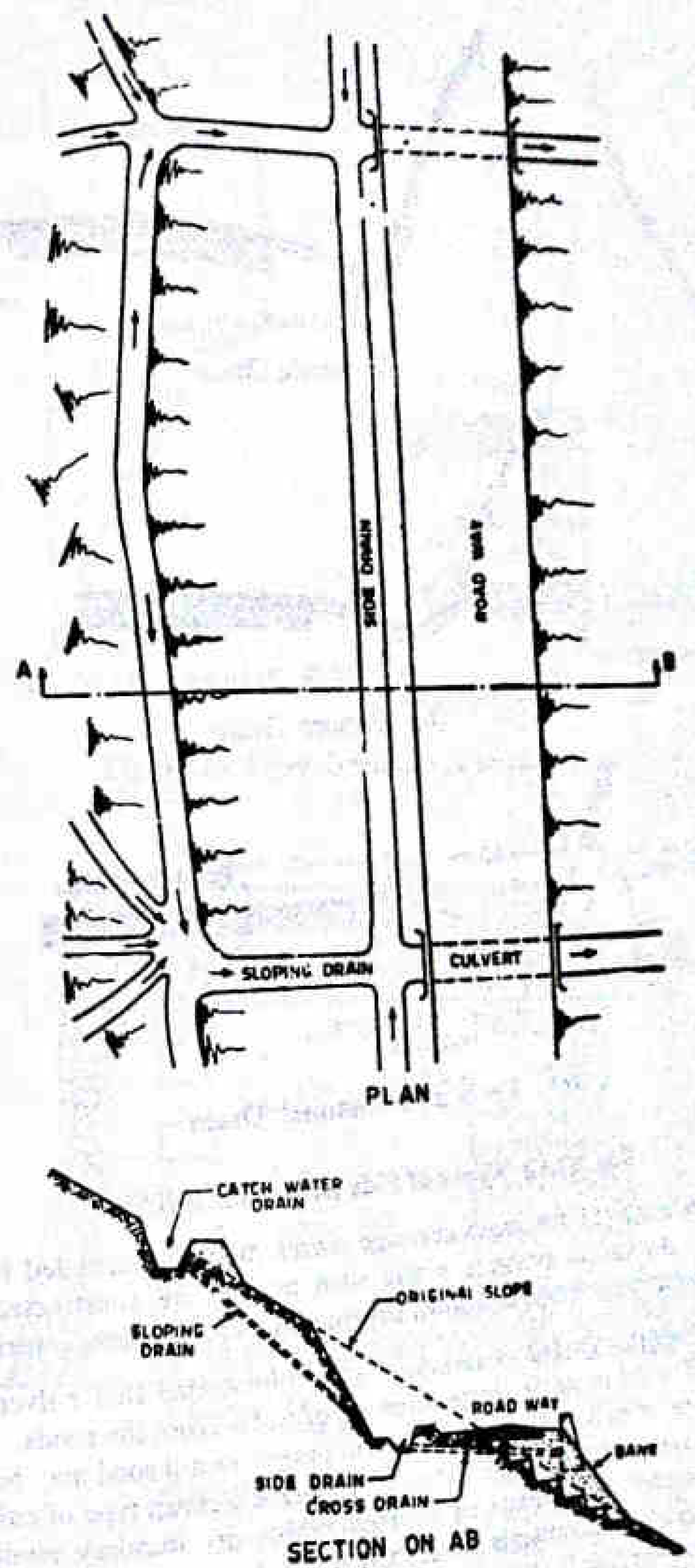
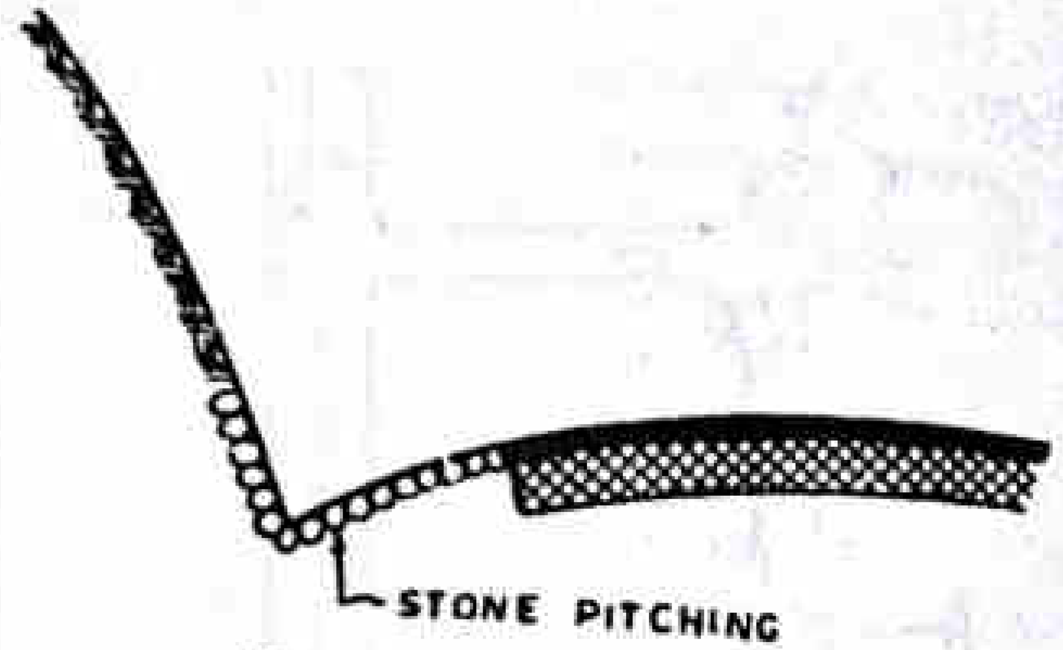
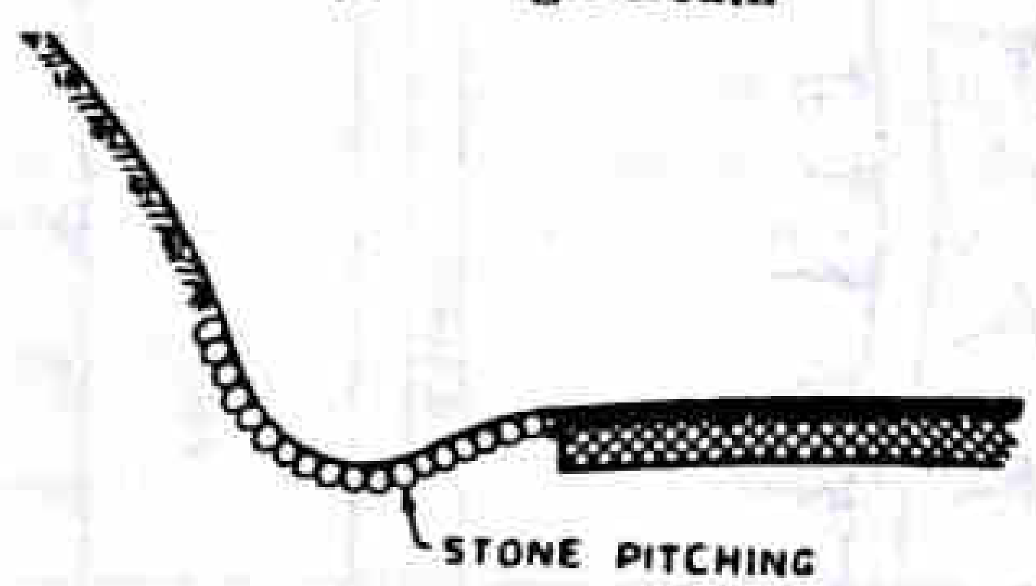


Fig. 12.11 Layout of Drainage System

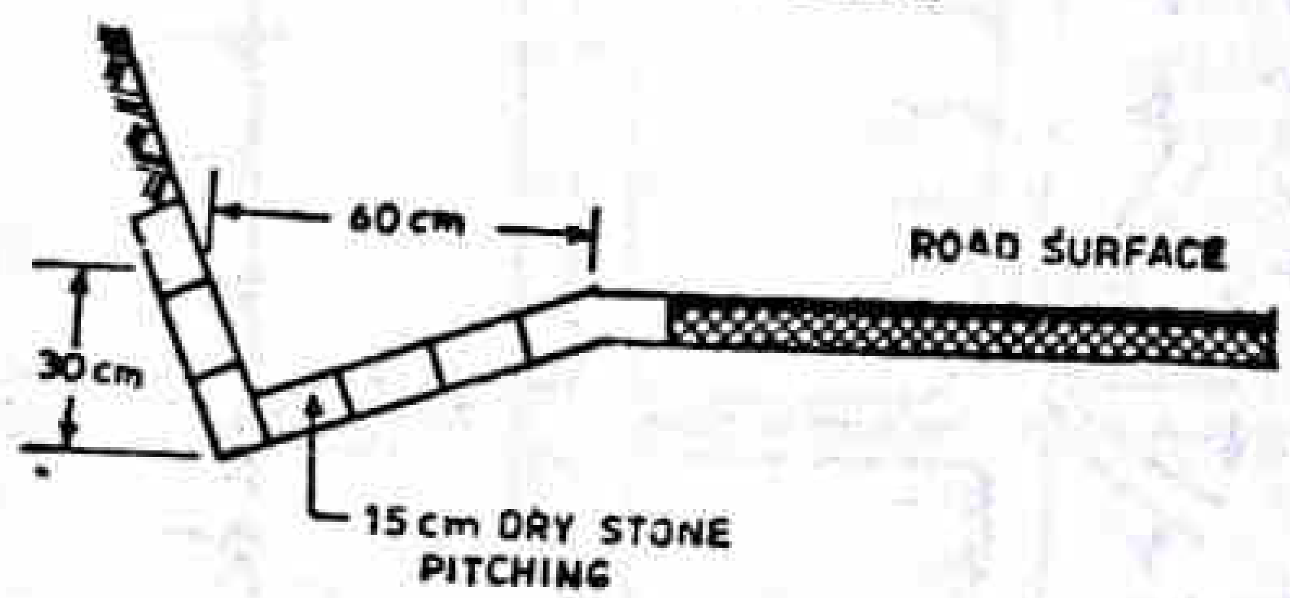
rubbish and to prevent scour. In rocky cuts catch pits can be provided as inlets in rocks as shown in Fig. 12.13. The floor level of the catch pit is deeper than the sill of the culvert or cross drain by at least 0.3 m. The floor of the cross drain is given a longitudinal slope of 1 in 7.



(a) Angle Drain



(b) Saucer Drain



(c) Kerb and Channel Drain

Fig. 12.12 Types of Side Drains on Hill Side

The waterway required for cross drainage works may be calculated by any one of usual formulas. As far as possible single span bridges are constructed. Protective structures are provided upstream and down stream of the cross drainage work.

In hill roads where rain-fall is heavy, it is recommended that culverts should be provided every 60 to 90 m, to facilitate drainage of water cross the roads. This may be quite costly. Hence often 8 to 10 *scuppers* per kilometer of hill road may be provided in addition to the bridges and regular culverts. A scupper is a cheap type of culvert or cross drain 0.9 to 1.0 metre wide, made of coursed rubble dry masonry abutments. See Fig. 12.14. The top of the abutments are corbelled with few layers of stone till the gap is 0.5 to 0.6 m, and a stone slab is laid on the top. Hand packed stones are placed on the top (0.3 to 0.6 m thick) and also around the scupper. Retaining walls are provided on both ends of the scupper.

Scuppers and *causeways* are usually provided for cross drainage in unimportant roads. Cause ways are submersible bridges, with a dip in the road profile to allow water flow across the road surface during floods.

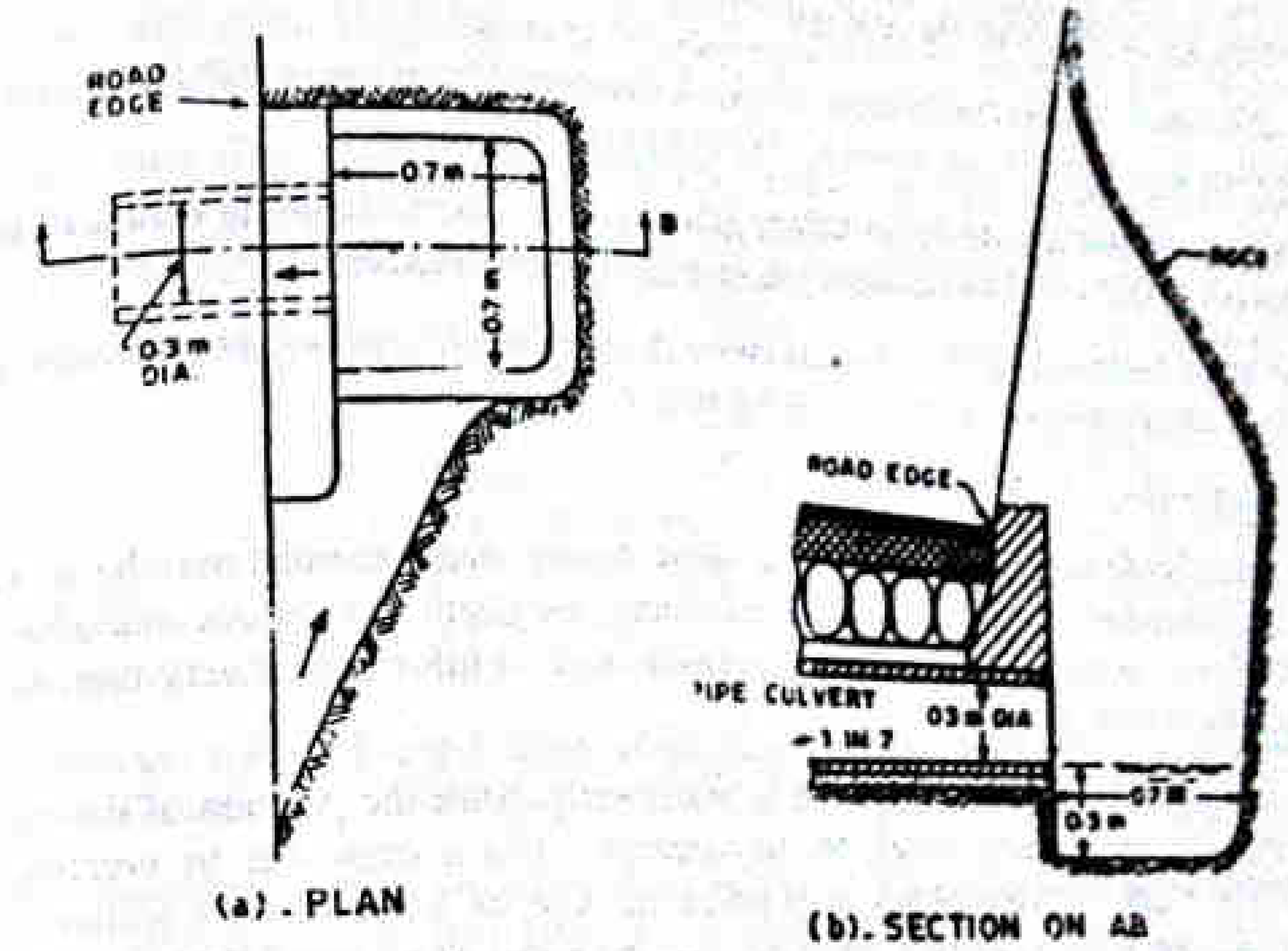


Fig. 12.13 Cross Drainage in Rock Cuts

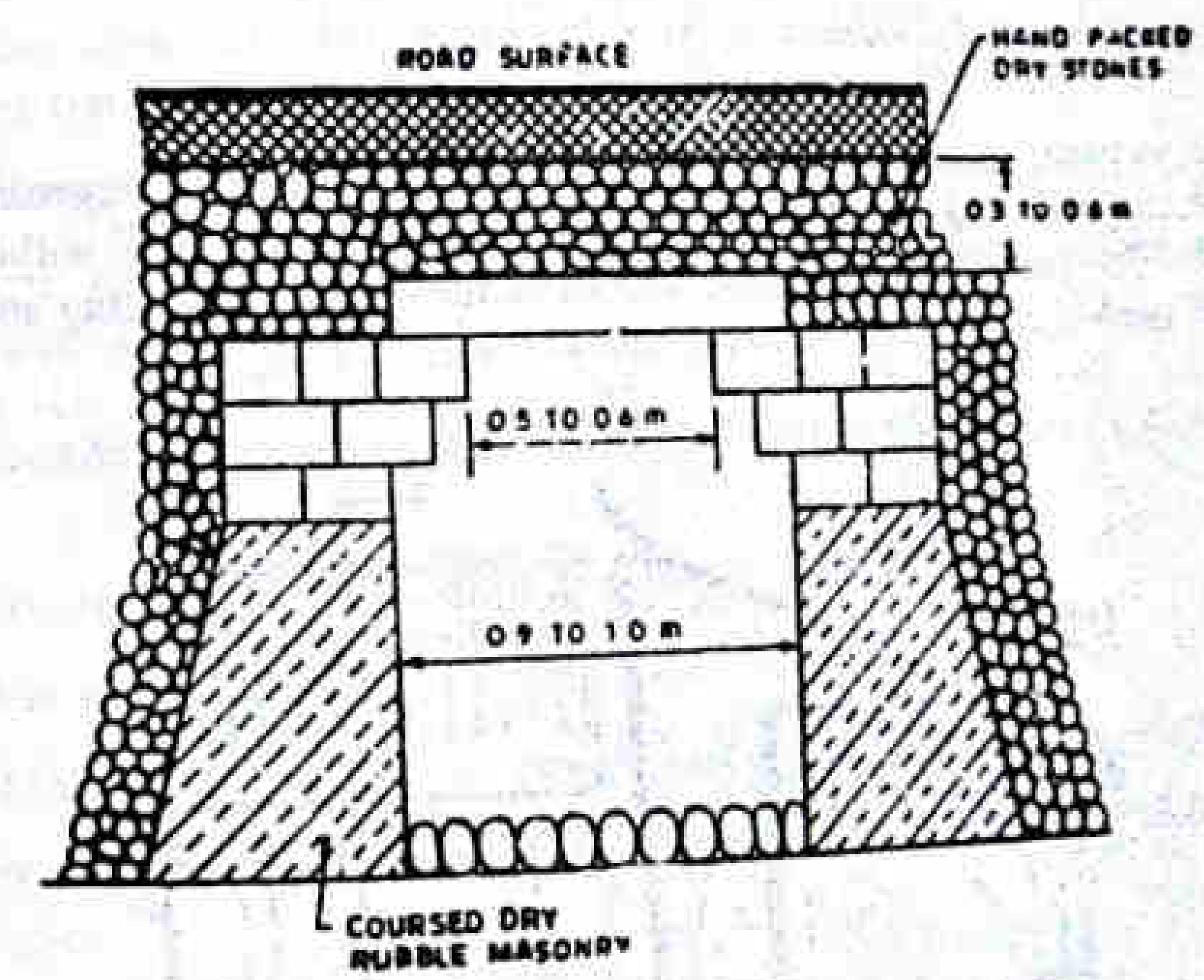


Fig. 12.14 Cross Section of Scupper

Sub-surface Drainage

The seepage flow of water on hill roads is one of the major problems during and after the monsoons. The seepage water may cut across the hill-side slope above, at or below the road level depending upon several factors such as depth of hard stratum and its inclination, quantity of under ground flow of water etc. The seepage flow causes problems of slope stability as well as the weakening of the road bed and the pavement. The seepage flow may be controlled by suitable sub-surface drainage system as shown in Fig. 11.7 under the chapter 'Highway Drainage'

12.6 MAINTENANCE PROBLEMS IN HILL ROADS

Maintenance of Drainage Structures

Catch water drains, side drains, catch pits and culverts are periodically cleared off of all blockages to prevent overflowing during rains.

Planting of trees on the upper slopes in order to reduce the scouring action of unstable ground due to rains is often resorted to as a precautionary measure.

The earth for repairing berms is taken by dressing the hill slopes into terraces, but the lowest terrace should not be more than 1.8 m in height.

Snow Clearance

The total accumulation of compacted snow during winter months may be as high as 6 m in the Himialayan region. At *avalanche* sides, the depth of snow accumulation could be much more. Because of snow accumulation most of hill roads at very high altitudes are closed for traffic in winter.

The first problem in snow clearance is to correctly locate the position of the road and other structures under snow cover, for movements. This is overcome by erecting *snow markers* which are wooden poles with metre marked on it, before the winter. Snow clearance is done with machines; but care must be taken when using them on black top surfaces not to damage them. Wheel dozers, snow blasts, motor graders or manual labour are used. If the ice crust is thick, blasting by explosives may be done without affecting structures.

Control of Avalanches

On some sections, special protective structures called galleries are constructed above the road which permit the snow mass to slide over the gallery roof without inducing impact loads. The galleries are therefore located on the bench of a cutting and the roof is covered with earth in order to continue the natural slope. See Fig. 12.15.

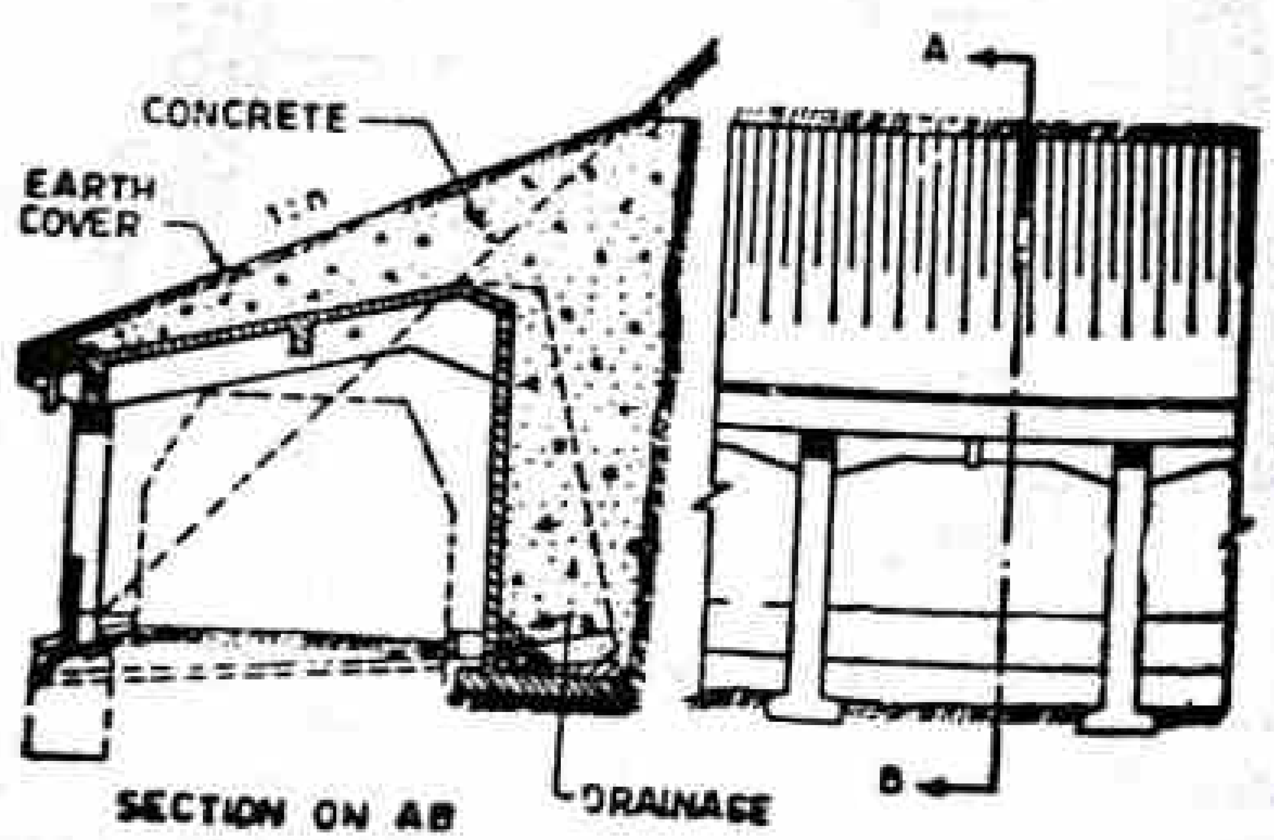


Fig. 12.15 R.C.C. Gallery for Breaking Avalanche

Sometimes, deliberate artificial release of snow is done by detonating explosives at the tension zone of the avalanche slab which brings down the snow in pieces. This method gives adequate warming and also minimizes the damages that can cause to the road structures.

Prevention of Land Slides

Land slides and slips are most important problems in the maintenance of hill roads. To tackle this, the engineer has to study the causes, correction and remedial measures. The subject by itself is very vast. However, it is briefly dealt with here.

The term land-slide denotes downward and outward movement of slope forming materials composed of natural rock, soils, artificial fills or combinations thereof. Land slides move along surface of separation by falling, sliding and flowing.

Land slide is classified into :

(a) falls

(b) slides

(c) flows and

(d) complex land slides.

Falls include free fall and rolling of rocks and debris without interaction of one fragment with another.

Slide is movement caused by finite shear failure along one or several surfaces which are visible or whose presence may reasonably be infered.

Flow is the movement within displaced mass such that the form taken by the moving materials resemble that of viscous fluid and the slip surface can not be located.

Complex land slide : In this, movement is by a combination of one or more of the types of failures stated above.

Factors Producing Slide Movements

Where shear stresses exceed the shear strength of the soil, movement occurs. Instability results only when shear failures have occurred at a number of points in a soil mass more or less constituting a surface along which a slip can occur. Anything that contributes towards a decrease in shear strength or an increase in soil stress can cause a slide.

Some causes of increased stress condition are listed below :

(i) Increase in water content.

(ii) External loads due to traffic, increase in weight due to accumulation of snow.

(iii) Removal of part of mass by excavation, removal of retaining wall and increase in slope angle.

(vi) Undermining caused by excavation or erosion.

(v) Shocks and vibrations caused by earthquakes or blasting.

Some causes for decrease in strength are as follows :

(i) Increase in water content and consequent swelling and increase in pore water pressure.

(ii) Hair-cracking due to alternate swelling and shrinkage of the soil structure.

(iii) Disturbance of intergranular bonds by vibration and formation of faults in bedding planes of strata.

(iv) Fissuring of pre-consolidated clay due to release of lateral restraint in a cut and buoyancy in saturated state decreasing effective intergranular pressure.

(v) Seepage pressures of percolating ground water resulting from viscous drag between the liquid and the soil grains.

Techniques of prevention and correction of land slides are listed below :

(a) Relocation of the highway.

(b) Effective drainage measures to intercept and divert water.

(c) Construction of buttress at toe and providing retaining structure.

(d) Slope treatment to minimise the erosion, and to improve the stability conditions.

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PROBLEMS

1. Explain why design, construction and maintenance of hill roads need special considerations.
2. What are the special points to be considered in the alignment of hill road ? Discuss.
3. Specify the details of geometric design and standards of hill roads including hair pin bends.
4. Explain briefly various problems in hill road construction and how they are overcome.
5. Discuss the importance of hill road drainage. With the aid of neat sketches, show the surface drainage system for effective drainage and disposal of water.
6. Design all geometric design elements of horizontal and vertical alignment of a hill road for a design speed of 50 kmph at (a) horizontal curve of minimum radius 90 m and (b) vertical summit and valley curves formed due to change in gradient equal to 9.0%. Assume all other data suitably.

7. Discuss the maintenance problems in hill roads.
8. Write short notes on :
   (a) Resisting length
   (b) Hair pin bend
   (c) Precipice work
   (d) Scupper
   (e) Prevention of land slide

Chapter 13

Road Side Development

13.1 ENVIRONMENTAL FACTORS IN PLANNING AND DEVELOPMENT OF HIGHWAYS

While planning development of roads and transportation facilities, apart from aiming at operational efficiency, it is also necessary to consider the quality of the environment due to the development. It is important to prepare an Environmental Impact Statement at the planning stage itself. Good results could be achieved if the aesthetic aspects of the roads are given due consideration at the time of highway planning. Other aspects which deserve attention are planning road side avenues, preservation of natural scenery and water sources, reservation of recreational areas and provision of various road side amenities to the road users. The air pollution and noise pollution aspects are also to be considered.

In order to ensure better environment on new highways, environmental studies should be carried out considering all the above aspects. A multi-disciplinary approach may be needed to make a cumulative assessment of land suitability for highway development causing minimum adverse effects to the surroundings.

On existing highways, an environmental evaluation study may be undertaken to assess the quality of the environment and also to quantify the functional and aesthetic components. Two pressing problems faced on the existing highways in the country are : (i) encroachments and (ii) ribbon development. Suitable legislation and proper enforcement measures are needed to deal with these problems.

The factors to be considered in the environmental studies are :

(i) highway factors such as land width, carriageway, foot path, cycle track, kerbs, shoulders, median, sight distance, horizontal curves, intersections, vertical curves, bridge and other structures

(ii) highway operation factors such as control and regulation, signs, signals, islands marking and other traffic control devices

(iii) maintenance aspects like borrow pits, stacking of materials and equipment, etc. and

(iv) road user amenities such as parking facilities, by passes, road side plantations, utility lines and general amenities like fueling stations, restaurants etc.

13.2 ROAD SIDE DEVELOPMENT & ARBORICULTURE

Road side development deals with the development of aesthetic and other amenities of road and the abutting land or the right of way. Proper planning is needed for road side development right from the stages of preliminary surveys for highway alignment and during construction.

The following are some of the points to be considered for this :

(i) Consistent and smooth horizontal and vertical alignments.

(ii) Wide right of way and shoulders in rural highways. Wide right of way in urban areas to screen adjoining property by plantation.

(iii) Flat side slopes in embankment and cut, rounded to blend to original surface.

(iv) Suitable planting of road side trees and shrubs and proper maintenance.

(v) Turfing on side slopes and on shoulders of rural road.

(vi) Developing pleasant views and parking places.

Planting of trees on the road side, or the *road arboriculture* is one of the important aspects in road side development. Trees provided on both sides of urban and rural road serve the following purposes :

(i) to provide attractive landscape of road sides

(ii) to provide shade to the road users

(iii) to protect against moving sand in desert areas

(iv) to provide fruit bearing trees and timber

(v) to intercept the annoying sound waves and fumes from road vehicles.

In urban areas, the road side planting is mainly for the beauty or the landscape and therefore, tests of ornamental and flowering species are generally preferred. On wide urban roads, the planting of shrubs is done on the medians or separators besides providing trees on road sides. These shrubs reduce the head light glare during night driving. Typical arrangement of road planting is shown in Fig. 13.1.

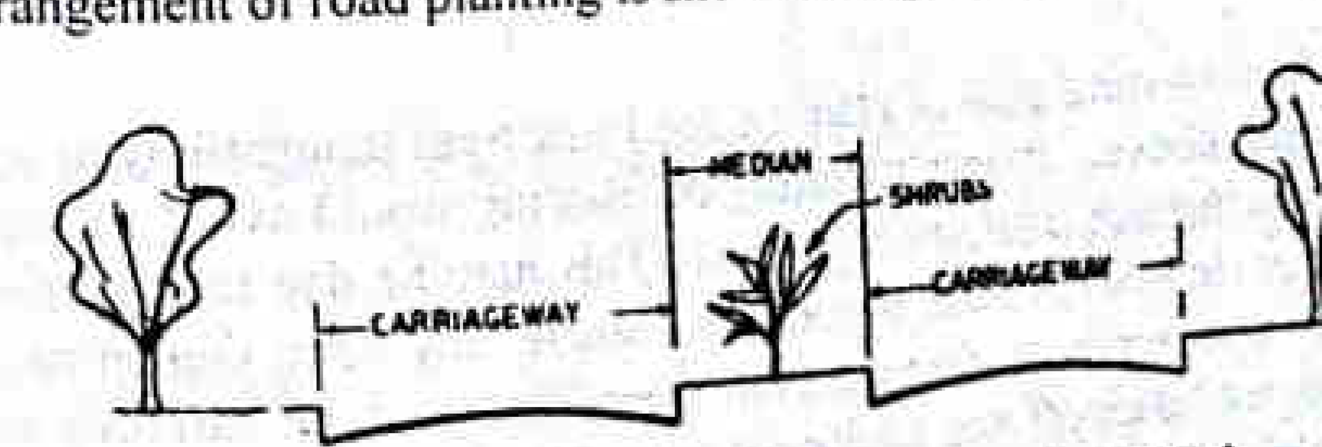
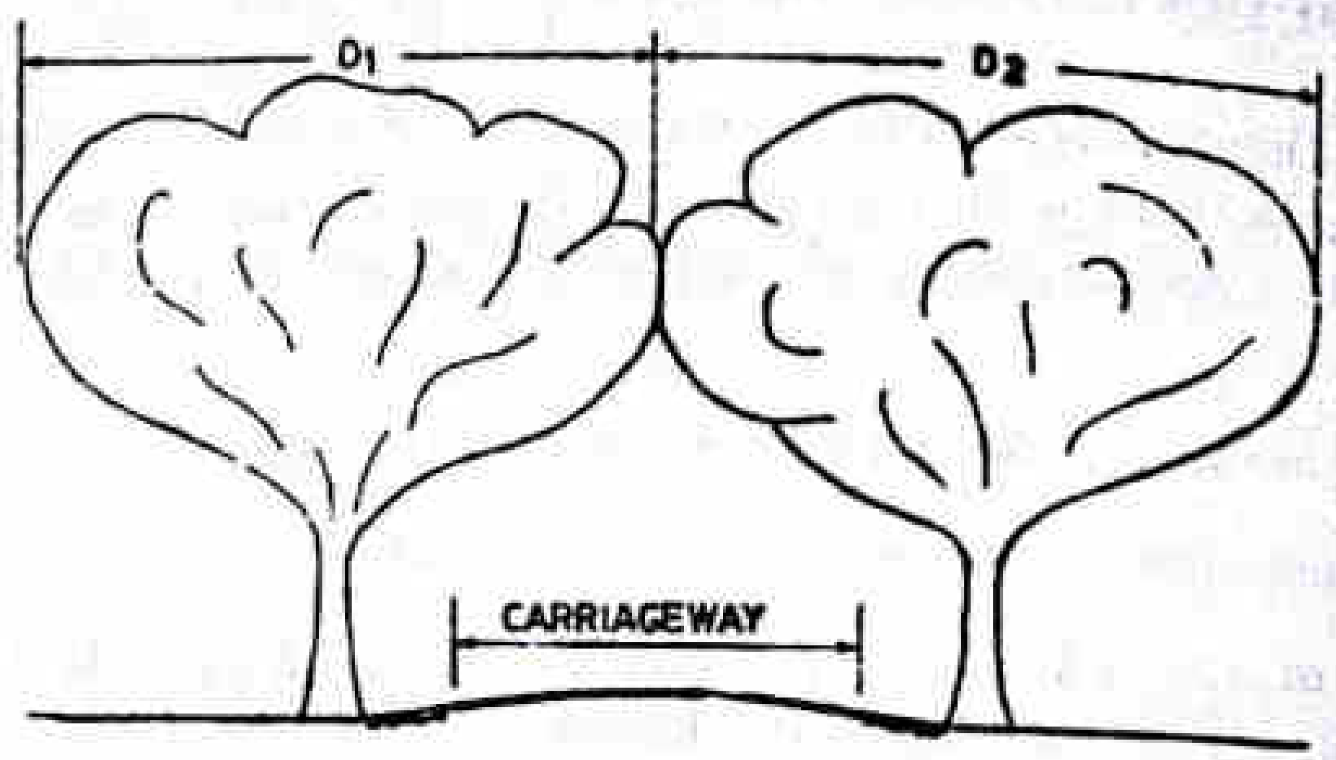
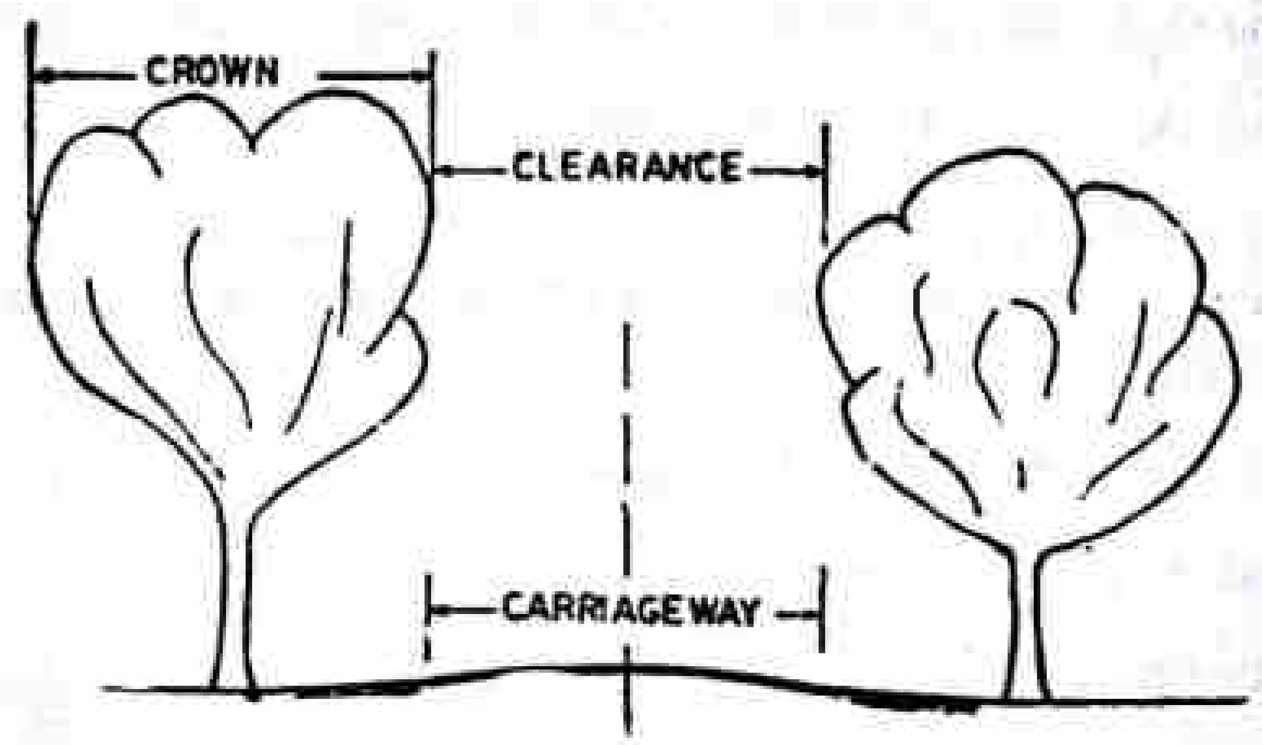


Fig. 13.1 Trees and Shrubs on Urban Road

Wide crowned trees on road sides are gentally not preferred, because they obstruct the day light and make the roads appear unsafe during night. It is desired that the crowns of the trees planted on both sides of a road do not cover the complete carriageway. (See Fig 13.2a). Arrangement shown in Fig. 13.2a is obviously not preferred as it covers the carriageway. It is recommended that the trees should be so planted on roadsides that the crown of trees do not extend beyond the pavement edges as shown in Fig. 13.2b. It is



(a) Carriageway Fully Covered



(b) Carriageway Clear to Sky

Fig. 13.2 Types of Tree Crowns

desirable that the trees should be at least 12 metre away from the centre of the road or carriageway.

13.3 PLANNING PLANTATION OF TREES

It is necessary to draw out a plan of planting road side trees thoughtfully to achieve the purposes as indicated above. Proper planning of planting would avoid wastage due to deterioration of trees through diseases or *felling*. This may be due to poor planning or arrangements. Many varieties of trees become real shade only when they grow for about 15 to 20 years. It is necessary to frame a plan where the trees are carefully examined; dead and ugly trees are taken off and additional trees are planted as a part of *renovation phase*. Trees are planted normally at about 12 meter apart longitudinally. In between this distance, additional trees are planted in an existing layout under the renovation phase as shown in Fig. 13.3. In preparing the plan, following points are considered :

(i) Each section of road is taken as an independent unit. Annual planting section is taken for about 3 to 4 km length. Blank or poor sections are assigned priority.

(ii) Section with *overmatured* and *established* trees are assigned the next priority. *Felling* and *renovation* are also taken up for these sections.

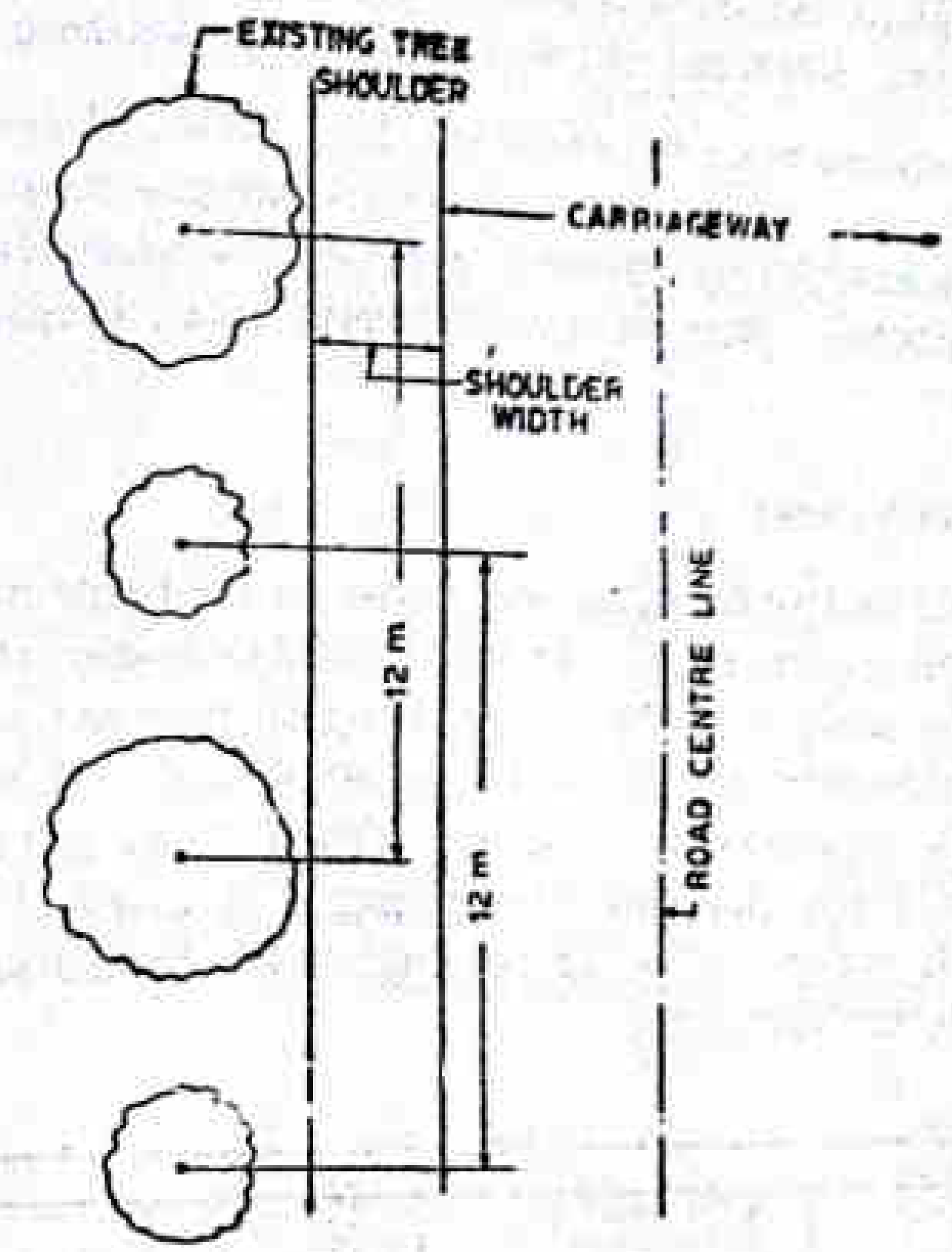


Fig. 13.3 Arrangement of Roadside Trees and Renovation

13.4 SPECIES AND THEIR SELECTION

13.4.1 Choice of Species

Species are selected on the basis of following considerations :

(i) Species should be such as to provide a large and dense crown.

(ii) Species should be easy to establish, fast developing and should be strong to resist the heavy storms and strong winds.

(iii) Species should be able to withstand *lopping* and *pruning*.

Types of Species

Species are selected depending on the factors like :

(i) Soil types

(ii) Watering conditions

(iii) Locations

(iv) Rain fall and other climatic factors.

Based on the above factors following species are generally preferred :

Based on soil types. The suitability of different species is judged on the basis of existing plantations etc. For *clayey soils*, Jamun, Mango etc. are recommended. In *sandy soils*, Shisham Kanji, Kanju and Arroo are suitable. In *loamy soils*, almost all types of species are suitable for growing.

In water - logged areas, Jamun, Arjun and Tarcharbi are found suitable. In *usar areas*, it is difficult to plant trees; Neem and Imli are suitable species for these areas.

Excellent avenues can be made from Shisham, Imli, Jamun, Mango, Arjun, Tun etc. These species require proper *protection* and frequent watering in the initial period. Near every kilometer stones on highways, flowering species viz. Amaltas, Kachnar, Gulmohar, Ashok etc. may be planted. Near wells and camping areas, Bargad and Peepal are planted.

13.4.2 Development of Nursery

Nursery is a plot of land where plants are grown for road side planting. The soil conditions, watering and climates etc. are carefully studied to develop sturdy plants out of the selected species for using them for final planting on the road sides. Number of nurseries are therefore required for different sections of roads. A nursery should be conveniently located for inspection and for transplantation. Since soil type and watering conditions control the health and growth of plants, it is essential to control these conditions favourable to the growth of the planting. Well drained sandy loam soil is considered very suitable for this purpose.

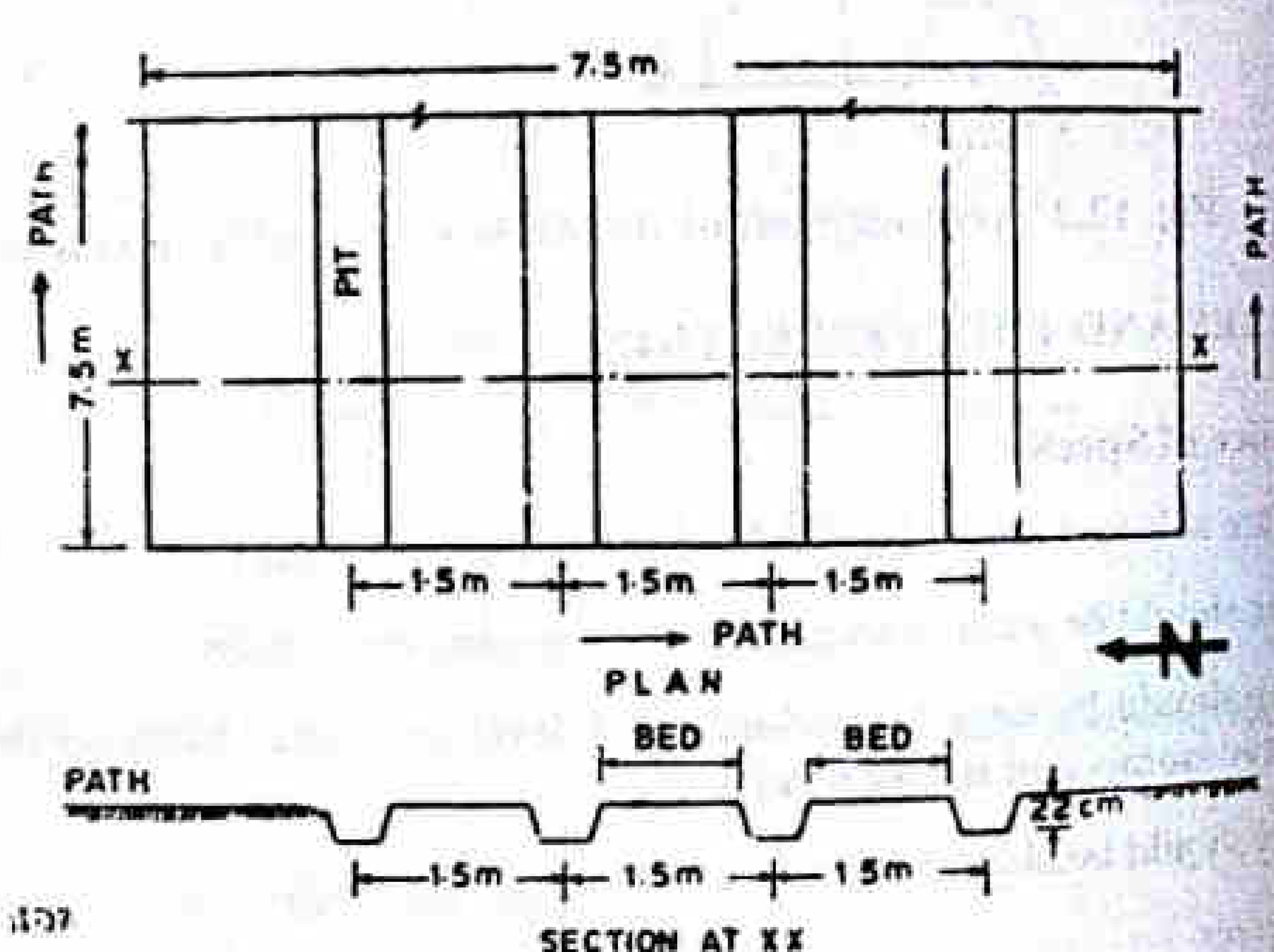


Fig. 13.4 Typical Layout of Nursery

Best manure is prepared either in compost pits or form a mixture of *ammonium sulphate* and *bone-meal* or *superphosphate of lime*. Normally, the equal proportions of these chemicals are used to form a mixture. The manure is applied when *seedings* have come up to 15 cm height. A nursey is established 2 to 3 years in advance for transplantation. It is recommended that the following number of plants are made available for the use :

Locations	No. of plants needed per km of road length	
	Site	Nursery
New avenues	180	270
Old avenues	100	150

Excess number of plants required at nursey is to account for seedling which may die out during their early growth and soon after transplanting.

13.4.3 Planting of Trees on Road Sides

The necessary locations are prepared at site at the transverse and longitudinal spacings as shown in Fig. 13.2b & 13.3. The pits for planting are prepared in the manner shown in Fig. 13.5. Sturdy and healthy plants are selected which are at least 1 to 1.5 metre high and have the stems of 2 cm diameter. Species like Shisham attain these dimensions in one year where as Imli, Neem, Jamun and Mango take about two years to attain this size in the nursery. It is recommended that only one plant should be placed in each pit. Sufficient manure is placed at the bottom of the pit before placing the seedling in position. The seedling need proper *protection* during their growth. Necessary protection is provided by

(i) Iron or R.C.C. guard

(ii) Brick work

(iii) Used (bitumen) drum

(iv) Trench

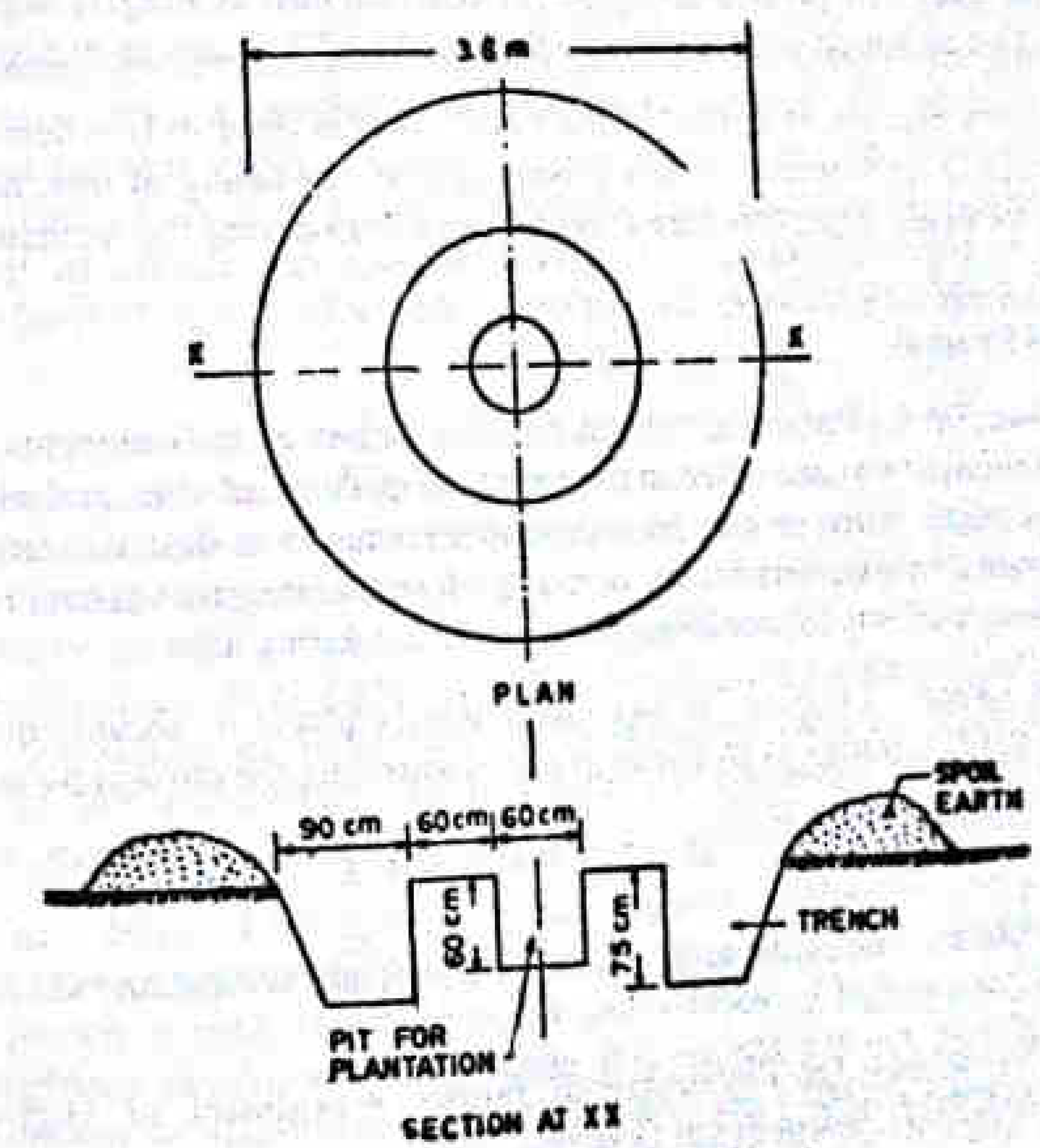


Fig. 13.5 A Pit for Planting Tree

Use of trench is illustrated in Fig. 13.5. This is the most effective method and is recommended for use in rural areas. For urban areas any one of the first three methods can be used depending upon the availability of funds.

Depending upon the soil types, plant and climatic conditions, the need and amount of watering is decided. Normally care is required in this respect for first two to three years, after which most of the trees are well established. Where protection is done through iron guard or similar devices, *earthen pitchers* are placed buried in the ground to provide water to the plant. The water is allowed to trickle only. The pots are filled in as and when required.

13.5 CARE OF TREES

Following operations are carried out on trees for their well being throughout the life span :

(i) Pruning and deforking

(ii) Lopping

(iii) Felling

Pruning is required for plants after they are placed in position. This process helps them to be more dense and growing straight. Pruning is done in cold weather only. Sharp knife or a pair of scissors is used for this purpose. The wounds are covered with bitumen coating. The cut is made vertical and the *bark* is not peeled off in the pruning process.

Lopping is done to drain the trees to develop a certain *shape* during its growth. This is a process by which extra branches of tree are removed which otherwise interfere with traffic or other trees in the vicinity. Branches are cut carefully, flush to the surface with trunk of the tree. To prevent *decay* of the tree, bitumen coating is applied on the cut portions. This operation is done either in the months of February or in September.

Felling is a process of removing the trees. This is done only when there are over-matured trees or trees which are too closely spaced. In felling of tree, the earth around the tree is removed, upper roots are exposed and cut observing the inclination of the stem of the tree.

Control of Erosion

Providing Turf. Providing turf on the side slopes of embankments, cuts and side drains and shoulders gives a pleasing appearance of the road side, and also protects the soil from erosion. There is also an appreciable reduction in dust nuissance on the road side. However turf shoulders are not always preferred as the grass cannot resist the traffic and there is a possibility of shoulders getting softened during rains.

Ground Cover. Growing herbaceous or woody plants or shrubs, not more than a metre in height at maturity, is an alternative for controlling the erosion by water and wind.

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PROBLEMS

1. Explain the objects or road side development.
2. What are the precautions to be taken in planning the plantation of road side trees? Discuss.
3. Write a note on development of a nursery.
4. Discuss the special maintenance and care required for road side planting.

Chapter 14

Highway Economics & Finance

14.1 INTRODUCTION

Better highway system provides varied benefits to the society. Improvements in highway results in several benefits to the road users such as : (i) reduction in vehicle operational cost per unit length of road (ii) saving in travel time and resultant benefits in terms of time cost of vehicles and the passengers (iii) reduction in accident rates (iv) improved level of service and ease of driving (v) increased comfort to passengers. Therefore the level of service of a road system may be assessed from the benefits to the road users.

The improvements in road net work also benefits the land owner by providing better access and consequently enhancing the land value. The cost of improvements in the highway fo land, materials, construction work and for the other facilities should be worked out. From the point of view of economic justification for the improvements, the cost reductions to the highway users and other beneficiaries of the improvements during the estimated period should be higher than the investments made for the improvement. In the planning and design of highways there is increasing need for analysis to indicate justification of the expenditure required and the comparative worth of proposed improvements, particularly when various alternatives are being compared.

Thus any new proposal for highway improvement or development should be justified in terms of cost incurred and benefits derived. In other words there should be economic justification for any expenditure incurred for improvement of highway facilities. Before finalising a highway development programme, it is necessary to compare the economy of various alternatives of alignment, pavement surfaces etc. The economic analysis is the only means for finalising the proposal and convincing the public about the need for certain investment.

Highway economy studies should be carefully planned to provide reliable inputs. Some of the important aspects are the forecasting of future consequences due to the investments in highway improvements and the comparison of various alternate proposals.

The government or any other agency finances highway developments. The funds for these are generally recovered from the road users in the form of *direct* and *indirect* taxations. *Highway Finance* deals with various methods of raising and or providing money for the highway projects.

14.2 HIGHWAY USER BENEFITS

14.2.1 General Benefits

Several benefits are brought to highway users and others due to the construction of a new highway or by improving a highway. Road user benefits are the advantages, privileges or savings that accrue to drivers or owners through the use of one highway facility as compared with the use of another. Benefits are measured in terms of the decrease in road user costs and the increase in road user services. Road user services are the advantages or privileges accruing to the vehicle driver or owner through features of safety, comfort, convenience etc. In some cases these can be evaluated in rupees per vehicle-kilometre.

A reduction in transportation cost as a result of highway improvements, would result in the reduction in the cost of almost every commodity which is transported from place to place before being delivered to the consumer. The impact of transportation cost on the cost of various commodities is conspicuous when there is an enhancement in transportation cost due to any reason, such as hike in price of petrol and diesel.

The various benefits due to highway improvement may be classified into two categories : (i) quantifiable or tangible benefits in terms of market values and (ii) nonquantifiable or intangible benefits.

Quantifiable Benefits

Various benefits which can be quantified include benefits to road user such as reduction in vehicle operation cost, time cost and accident cost. The other benefits include enhancement in land value. These are briefly explained below :

(i) Saving in vehicle operation cost is due to reduction in fuel and oil consumption and reduction in wear and tear of tyres and other maintenance costs. A road with sharp curves and steep grades require frequent speed changes; presence of intersections require stopping idling and accelerating; vehicle operation on road stretches with high traffic volume or congestion necessitates speed changes and stoppings and increased travel time; all these factors result in an increase in every component of vehicle operation cost. Uneven pavement surface condition with ruts, pot holes, undulations, waves and corrugations increases the vehicle operation cost due to increase in fuel consumption, tyre wear and the general maintenance cost of the vehicles. (Apart from this, there is considerable increase in fatigue and discomfort to drivers and passengers due to travel on such bad road surfaces, which cannot be quantified).

(ii) Saving in travel time is of direct consequence to commercial vehicles due to possible increase in their trip length and earning per unit time. Benefits due to saving in travel time may be assigned in terms of time cost of vehicles.

(iii) Value may also be assigned for the saving in travel time of passengers. A part of the time saved by the passengers or commuters may be used for some useful purpose and a value can be assigned for the saving in travel time.

(iv) The reduction in accident rate due to improvements in the highway facilities causes considerable benefits to the road users and others. The component of accident costs may include cost of damages to vehicles and other properties, injuries and loss of human life, delays to vehicles and the passengers, cost for investigations, legal proceedings etc.

(v) The benefits to other than road users include the enhancement in land value increase in employment opportunities and related economic uplift.

Non-quantifiable Benefits

The non-quantifiable benefits due to improvements in highway facilities include reduction in fatigue and discomfort during travel, increase in comfort and conveniences and improvement in general amenities, social and educational aspects, development of recreational and medical services, improved mobility of essential services and defence forces, aesthetic values, etc. Yet another important intangible road user benefit is the reduced suffering and pain of those involved in highway accidents.

14.2.2 Motor Vehicle Operation Cost

The factors to be considered for evaluating motor vehicle operation cost would differ depending on the purpose of the analysis. The vehicle may be classified in different groups such as passenger cars, buses, light commercial vehicles, single unit trucks, combination vehicles etc., for the purpose of cost analysis. The motor vehicle operation costs depend on several factors which may be grouped as given below :

(i) Cost dependent on time expressed as cost per year such as interest on capital, depreciation cost, registration fee, insurance charges, garage rent, driver's license, salaries etc. as applicable.

(ii) Cost depending on distance driven expressed as cost per vehicle-kilometre. The items which may be included here are fuel, oil, tyres, maintenance and repairs etc.

(iii) Cost dependent on speed include cost of fuel, oil and tyre per vehicle-km-time-cost of vehicles, travel time value of passengers, etc.

(iv) Cost dependent on type of vehicle and its condition. Operation cost of larger vehicles are comparatively higher. The operation cost of old vehicles maintained in poor condition is also higher.

(v) Cost dependent on road condition and geometrics such as type and conditions of pavement surface, magnitude and length of gradients, radius and number of horizontal curves etc. The vehicle operation cost increases with the unevenness index of pavement surface. These factors are also affected by the topography of the region. On hill roads the vehicle operation cost is higher than on plains.

(vi) Cost dependent on traffic factor such as congestion, volume to capacity ratio, flow characteristics, composition of traffic etc.

(vii) Value of occupant's time and

(viii) Accident costs.

The costs of vehicle operation and time for unit distance may be taken as :

$$T = a + \frac{(b+c)}{\text{speed}} \tag{14.1}$$

where

a = running cost per unit distance, independent of journey time

b = a fixed hourly cost, dependent on speeds

c = the portion of the running cost which is dependent on speed

Therefore the operation costs may be considered to consist various components like motor fuel cost, lubricating oil consumption, tyre costs, vehicle repair and maintenance, depreciation, cost due to slowing, stopping, idling and standing delays, costs related to pavement surface and its condition, grades, curves and traffic volumes. Also the time costs and accident costs are taken into consideration. A major research project on 'Road User Cost Studies' was taken-up in India at the Central Road Research Institute, New Delhi with the assistance of the World Bank.

Some of the variable cost components which are dependent on speed are illustrated in the form of charts for hypothetical cases in Fig. 14.1 to 14.6. Figure 14.4 shows the relationship of fuel consumption with speed on various road gradients. There is an optimum speed for minimum fuel consumption for every motor vehicle. Figure 14.2 shows the relationship of tyre life with speed for some typical pavement surface condition. As the speed increases, the tyre wear increases and the tyre life decreases. Figure 14.3 shows the variation of time cost of vehicle with speed for different time values per vehicle-hour. Figure 14.4 shows the variation in additional operating cost of vehicle for stopped and unstopped delays for different approach speeds. Figure 14.5 shows the additional operating cost due to speed change cycles for different operating speeds. If similar charts are developed for typical road vehicles from extensive field data, the motor vehicle operation cost could be easily estimated for various conditions. Figure 14.6 shows variation of total running cost of vehicle with speed and the details of some component factors.

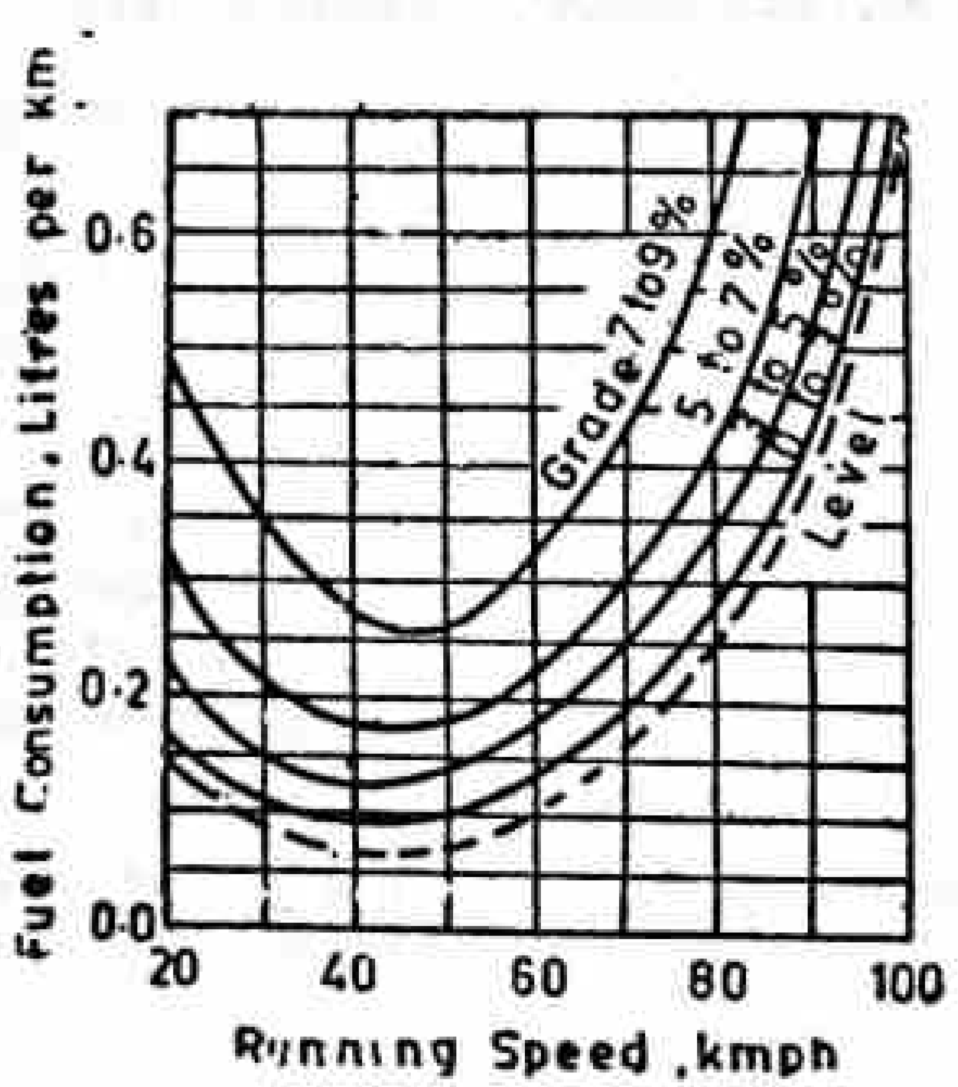

Fig. 14.1 Fuel Consumption Chart

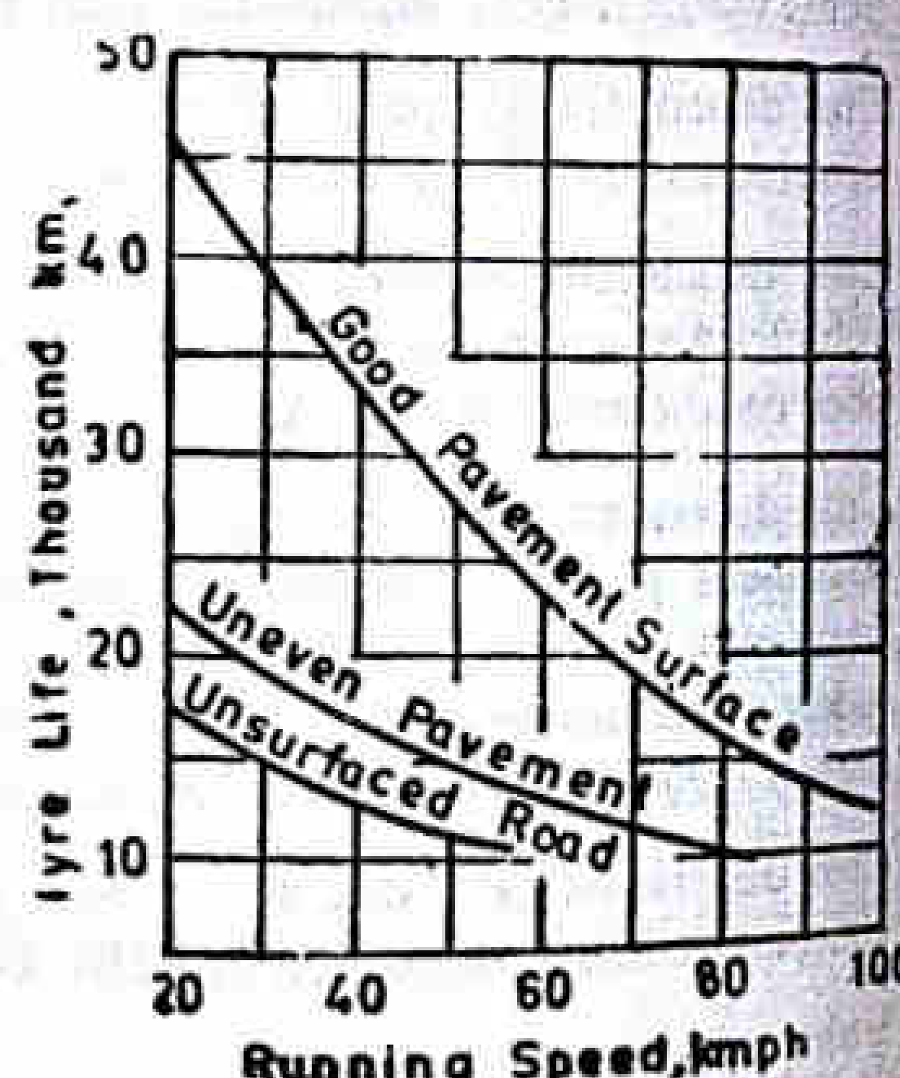

Fig. 14.2 Tyre Life Chart

Calculation of motor vehicle operation cost using the charts are illustrated in Examples 14.1 and 14.2.

Example 14.1

Calculate the operating cost of a passenger car for 100 km length of a rural highway with no sharp curves for most economical speed of vehicles operation using the following data and charts given in Figs. 14.1, 14.2, 14.3 and 14.4.

Gradients : level for 25 km, 0 to 2% for 55 km, 4% for 15 km and 6% for 5 km.

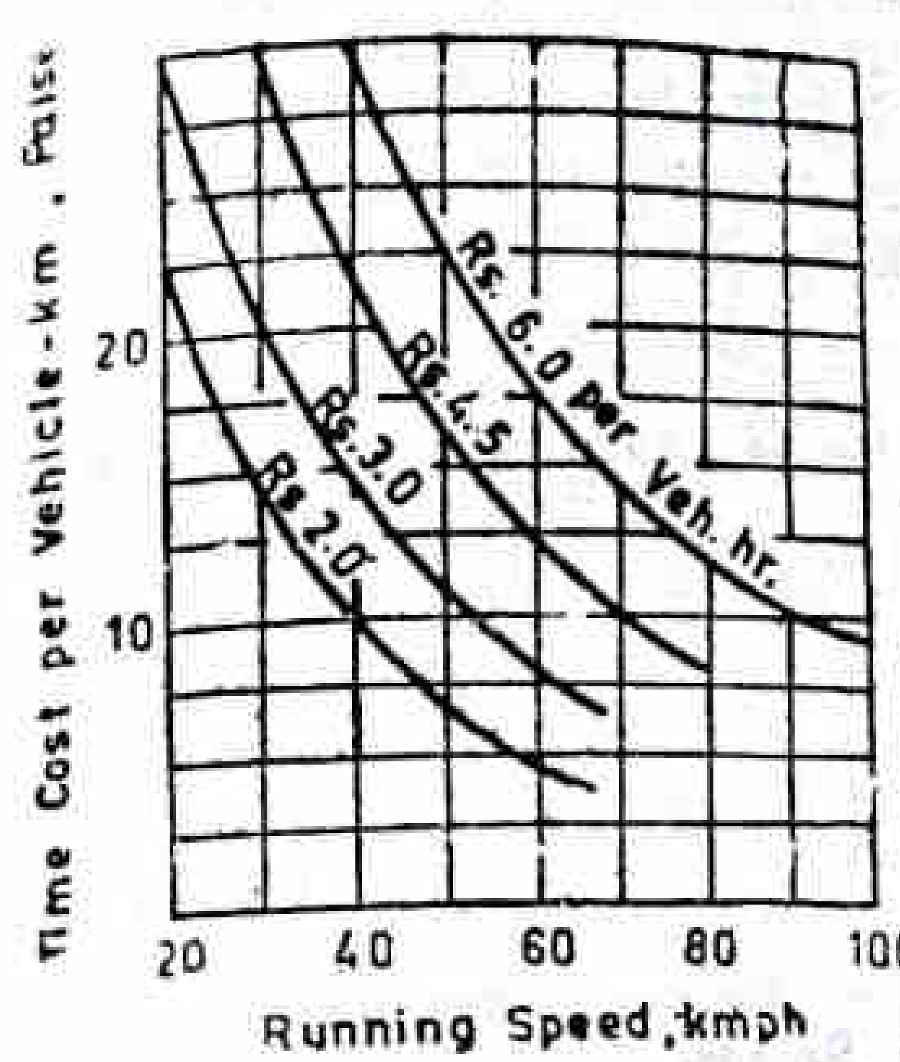

Fig. 14.3 Time Cost per Vehicle kilometre

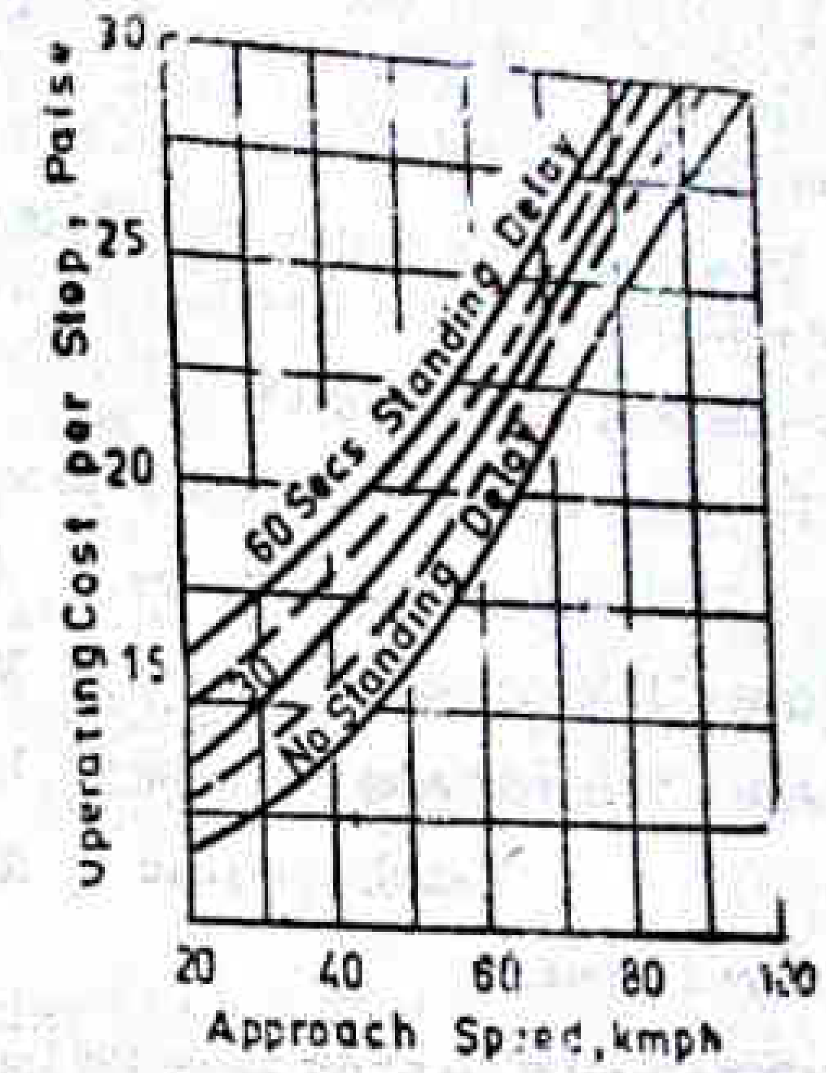

Fig. 14.4 Additional Operation Cost per Stop

Pavement surface condition : good

Fuel cost : Rs. 8.50 per litre

Tyre cost : Rs. 950/- per tyre

No. of stops enroute : 10 stops without delay, 5 with 45 seconds delay and 4 with 60 seconds delay

Time cost per vehicle hour : Rs. 3.00

Depreciation cost of vehicle : 5 paise per km

Solution

(i) Fuel Cost

Gradient %	Economical operation speed kmph	Length km	Fuel consumption lit/km (Fig. 14.1)	Total fuel litre
Level	42	25	0.07	1.50
0-2	42	55	0.09	4.95
4	40	15	0.12	1.80
6	45	5	0.17	0.85
		100		9.10

Total fuel consumption for 100 km length = 9.1 litre. (from table).

Total fuel cost = 9.1×8.50 = Rs. 77.35

(ii) Tyre Cost

Adopting an average economical running speed of about 42 kmph, the tyre life on good pavement surface (Fig. 14.2) = 32,000 km.

Cost of 4 tyres on 100 km. Stretch of Rs. 950/- per tyre.

$$= \frac{950 \times 4 \times 100}{32{,}000} = \text{Rs. } 11.88$$

(iii) Time Cost for Vehicle

Using the time cost curve of Rs. 3.00 per vehicle-hour (Fig. 14.3) the cost per vehicle-km at 42 kmph speed at 16 paise, time cost for 100 km = Rs. 16.00.

(iv) Additional Cost for Stopping

For average approach speed of 42 kmph using Fig. 14.4 additional operating cost for:

10 stops with no delay at 14 paise	=	Rs. 1.40
5 stops of 45 seconds delay at 18 paise	=	Rs. 0.90
4 stops of 60 seconds delay at 19 paise	=	Rs. 0.76
Total Stopping cost	=	Rs 3.06

(v) Depreciation Cost

Depreciation cost at 5 paise per km for 100 km = Rs. 5.00

(vi) Total Vehicle Operation Cost for 100 km

At economical operating speed = 77.35 + 11.88 + 16.00 + 3.06 + 5.00

= Rs 113.29

Example 14.2

The distance between two towns is 120 km. Calculate the average operation cost per vehicle-km for a desired average running speed of 60 kmph from the following data :

Fuel cost Rs. 8.60 per litre

Tyre cost Rs. 940.00 per tyre

Gradients = 5 km level, 55 km 0 to 3 percent grade and 10 km 3 to 5% grade

Pavement surface condition = good.

Speed reduction to 40 kmph at five curves and to 20 kmph to 8 congested zones.

Total 12 stops with average 60 seconds, delay and time cost per vehicle-hour = Rs. 4.50.

Depreciation cost of vehicle = 6 paise per km

Charts (Fig. 14.1 to 14.5) may be used.

Solution

(i) Fuel Cost at 60 kmph

Road details	*Fuel consumption, lit.*
Level 55 × 0.1 (Fig. 14.1)	5.5
0 to 3% grade 55 × 0.14	7.7
3 to 5% grade 10 × 0.18	1.8
Total fuel consumption	15.0 lit.

Fuel cost 15 × 8.60 = Rs. 129.00

(ii) Tyre Cost

Tyre life at 60 kmph on good surface (Fig. 14.2)

= 24,000 km

Tyre cost for 4 tyres @ Rs. 940.00 = $\frac{4 \times 940 \times 120}{24000}$ = Rs. 18.80

(iii) Time Cost of Vehicle

Time cost @ Rs. 4.50 per vehicle km at 60 kmph (Fig. 14.3)

= 12 paise per km

Cost of 120 km = 0.12 × 120 = Rs. 14.40

(iv) Speed Reduction Cost (Fig. 14.5)

From 60 kmph to 40 kmph = 4 paise × 5 = Rs. 0.20

From 60 kmph to 20 kmph = 10 paise × 8 = Rs. 0.80

Total = Rs. 1.00

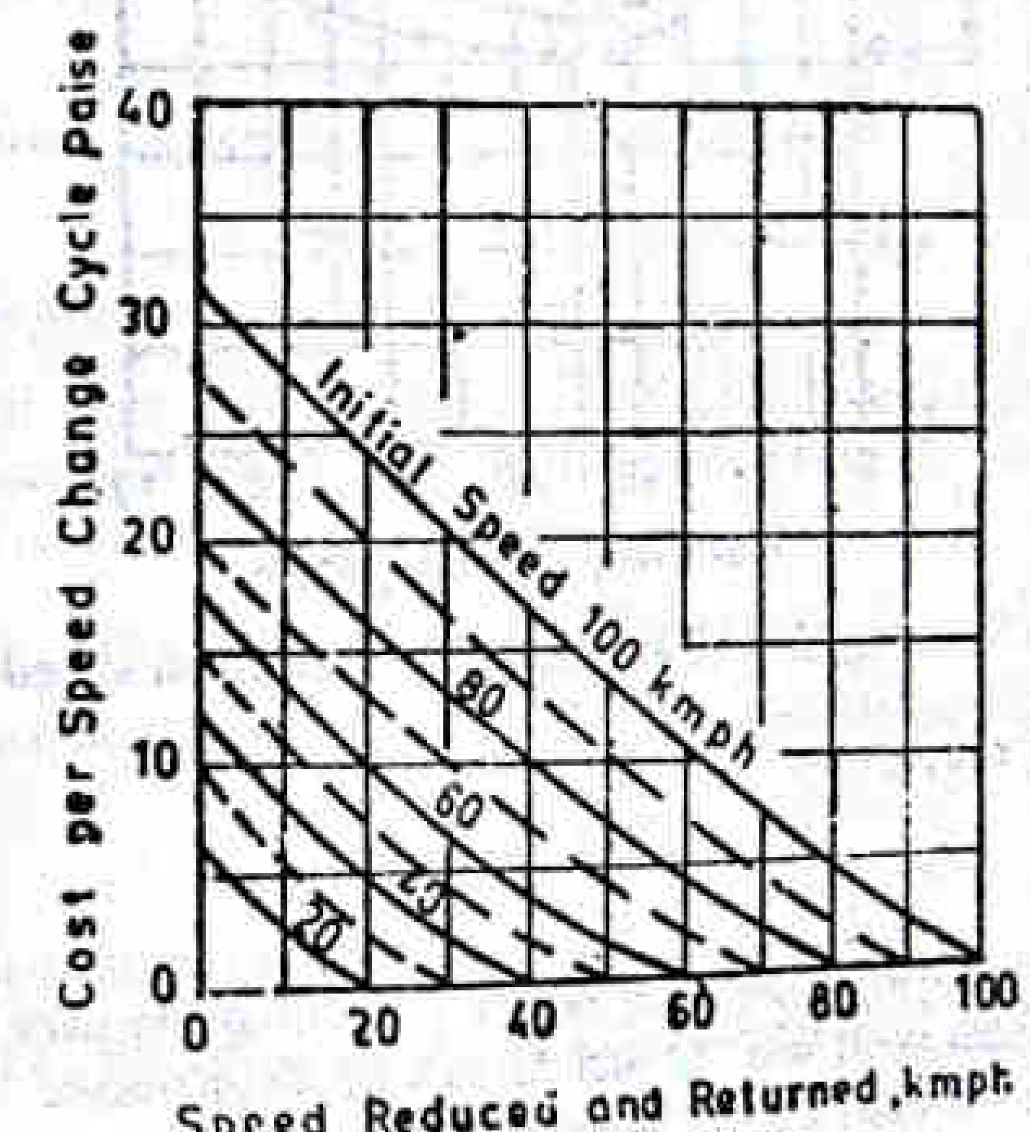

Fig. 14.5 Additional Operating Cost due to Speed Change Cycle

(v) Additional Cost For Stopping

From a speed of 60 kmph 12 stops of 60 secs. delay

(Fig. 14.4) = 24 × 12 = Rs. 2.88

(vi) Depreciation Cost

Depreciation @ 6 paise per km × 120 km = Rs. 7.20,

(vii) Total vehicle operation cost for 120 km length

= Rs. 129.00 + 18.80 + 14.40 + 1.00 + 2.88 + 7.20

$$= \text{Rs. } 173.28$$

$$\text{Average vehicle operation cost per km} = \frac{173.28}{120} = \text{Rs. } 1.44$$

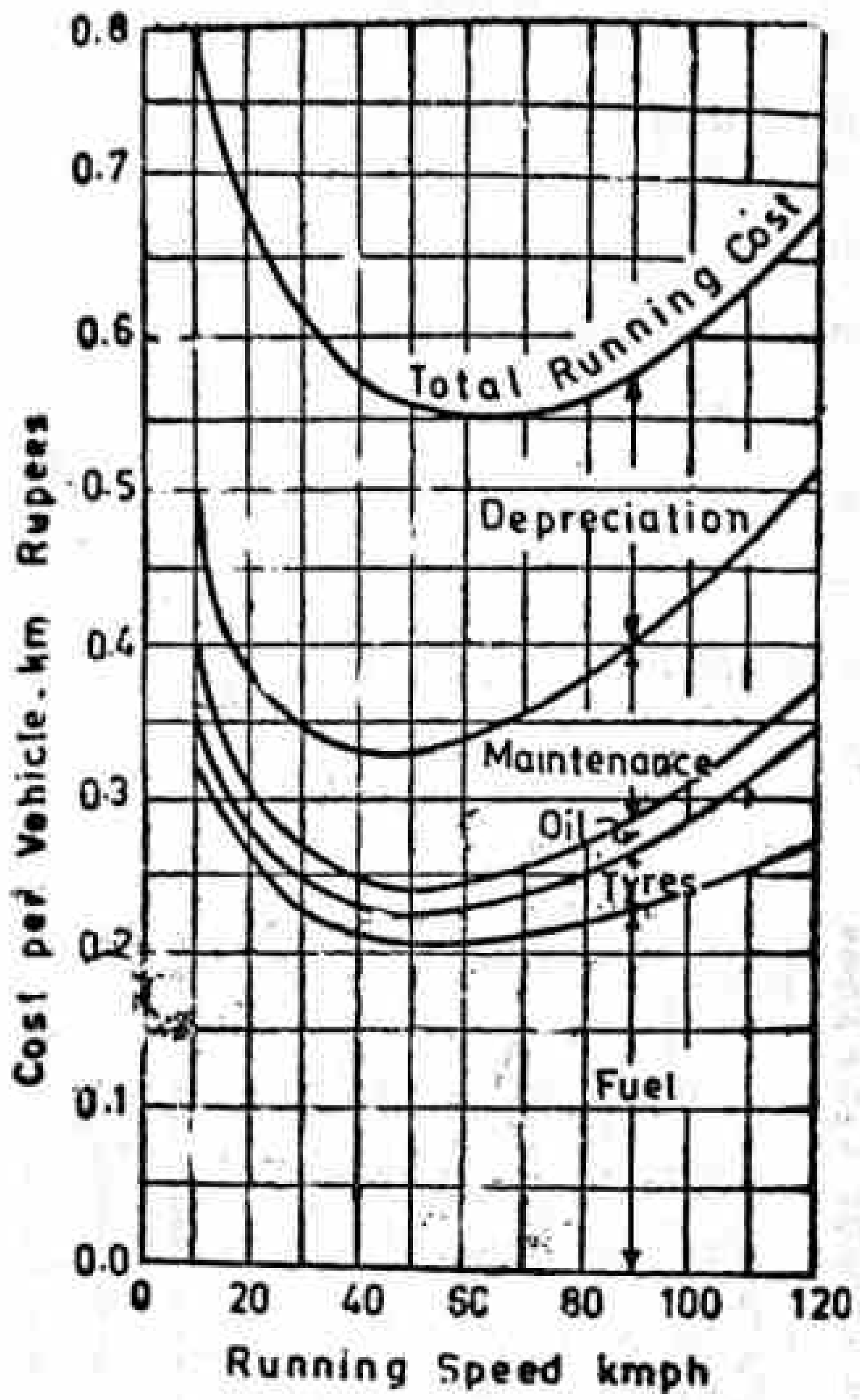

Fig. 14.6 Vehicle Running Cost (Passenger car at uniform speed on level tangent)

14.3 HIGHWAY COSTS

14.3.1 General

The total Highway Cost for road user benefit analysis is the sum of the capital costs expressed on an annual basis and the annual cost of maintenance. The total cost for highway improvement is obtained from the estimate prepared from the preliminary plans. The total cost of each highway engineering improvement proposal is calculated from the following five components :

(i) right of way

(ii) grading drainage, minor structures

(iii) major structures like bridges

(iv) pavement and appurtenances

(v) annual cost of maintenance and operation

Computation of total annual highway cost based on summation of the annual cost of individual items of improvements and their average useful lives is considered to be a proper and accurate approach.

It is difficult to estimate the service lives of highway elements as there are several variables such as soil, climate topography and traffic. Road life studies enable estimation of lives of pavements, bridges and other roadway facilities. The service life of cement concrete pavement may be assumed to be 20 to 40 years, that of bituminous concrete 10 to 15 years and other bituminous surfaces including surface dressing 3 to 10 year. The water bound macadam has a life of 2 to 6 years and the low cost surfaces 2 to 4 years. However, these are with the assumption that the total thickness requirement of pavement has been designed properly considering the design factors.

Generally a life of 75 to 100 years is assumed for land for right of way, 40 to 50 years for earth work grading and structures, 50 to 75 years for major bridges and 5 to 20 years for traffic control devices.

The cost of highway agencies towards construction and maintenance may be included in economic studies, where appropriate. Interest on capital is treated as one of the costs. The interest is either charged at the current rate at which the highway agency can borrow or at a rate representing minimum attractive return.

14.3.2 Annual Highway Cost

The items to be included while computing annual highway cost are :

(i) Administration (a portion) : Personal service, building, equipment operation, office, insurance etc.

(ii) Highway operation : Equipment, building vehicle operation including capital costs of vehicle.

(iii) Highway maintenance

(iv) Highway capital cost : Cost of highway components such as right of way, damages, earthwork, drainage system, pavement bridges and traffic services depreciation cost and interest on investment.

(v) Probable life and salvage value at the end of this period.

The average annual highway cost for a road system may be summed up by the formula

$$C_a = H + T + M + C_r \quad (14.2)$$

where C_a = average annual cost of ownership and operation

H = average cost for administration and management at head quarters.

T = average annual highway operation cost.

M = average annual highway maintenance cost.

C_r = average annual capital cost of depreciation of investment plus interest on capital or the *capital recovery* with return on capital

The annual cost is considered in the economic assessment of highway projects. Instead of considering the overall cost of a project the annual repayment of a capital loan plus the interest over a specified period of time of the *annual capital cost* is considered in the analysis. The first cost of a capital improvement is converted into equivalent uniform annual cost by the formula :

$$C_r = P\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] = P(CRF) \quad (14.3)$$

where C_r = receipt in a uniform series for n periods to cover P at a rate of interest i

P = first cost of improvement of an element of a highway

i = rate of interest per unit period

n = period of time in number of interest periods

CRF = capital recovery factor = $\frac{i(1+i)^n}{(1+i)^n - 1}$

At the end of the service life of road pavement, some of the items could be assigned some salvage value. However the salvage value of some other items may be negligible.

The average annual capital cost C_r for a project considering salvage value may be estimated by the use of the formula (for the capital-recovery with salvage value):

$$C_r = (C - V_s)\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] + i\,V_s$$

$$= (C - V_s)\,CRF + i\,V_s \qquad (14.4)$$

where C = total investment on construction

V_s = salvage value at the end of n years

i = interest rate applicable

n = number of years of expected use of the facility

The compound amount accumulated sum S on the principal sum of proposed improvement cost or single payment P, including interest rate, i in n years is given by:

$$S = P(1+i)^n \qquad (14.5)$$

Example 14.3

Calculate the annual cost of a stretch of highway from the following particulars:

Item	Total cost Rs. In lakhs	Estimated life, years	Rate of interest %
Land	12.0	100	6
Earth work	9.0	40	8
Bridges and culverts	7.5	60	8
Pavement	14.0	15	10

The average cost of maintenance of the Road is Rs. 1.5 lakhs per year.

Solution

$$\text{Annual cost } C_r = P\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] = P \times CRF_{(i,\,n)}$$

(i) Annual cost of land $= 12 \times \frac{0.06(1+0.06)^{100}}{(1+0.06)^{100} - 1}$

$= 12 \times CRF_{(i = 6,\ p.c.,\ n = 100)}$

$= 12 \times 0.06018 = \text{Rs. } 0.72216 \text{ lakhs}$

(ii) Annual cost of earthwork $= 9 \times CRF_{(i = 0.08,\ n = 40)}$

$= 9 \times 0.08386 = \text{Rs. } 0.75474 \text{ lakhs}$

(iii) Annual cost of bridges $= 7.5 \times CRF_{(i = 0.08,\ n = 60)}$

$= 7.5 \times 0.08080 = \text{Rs. } 0.606 \text{ lakhs}$

(iv) Annual cost of pavement $= 14 \times CRF_{(i = 0.1,\ n = 15)}$

$= 14 \times 0.13147 = \text{Rs. } 1.84058 \text{ lakhs}$

(v) Average annual maintenance cost = Rs. 1.5 lakhs

(vi) Total annual highway cost $= 0.72216 + 0.75474 + 0.606 + 1.84058 + 1.50$

$= \text{Rs. } 5.42348 \text{ lakhs}$

Example 14.4

Compare the annual costs of two types of pavement structures, (i) WBM with thin bituminous surface at total cost of Rs. 2.2 lakhs per km, life of 5 years, interest at 10%, salvage value of Rs. 0.9 lakhs after 5 years; annual average maintenance cost of Rs. 0.35 lakhs per km and (ii) Bituminous Macadam base and bituminous concrete surface, total cost of Rs. 4.2 lakhs, life of 15 years interest at 8%, salvage value of 2.0 lakhs at the end of 15 years; annual average maintenance cost Rs. 0.25 lakhs per km.

Solution

Annual average cost taking salvage value into consideration,

$$C_r = (C - V_s)\frac{i(1+i)^n}{(1+i)^n - 1} + i\,V_s + \text{average annual maintenance cost, M}$$

$$= (C - V_s)\,CRF_{(i,\,n)} + i\,V_s + M$$

(i) Annual cost of pavement with WBM base and thin bituminous surface course.

$= (2.2 - 0.9)\,CRF_{(i = 0.1,\ n = 5)} + 0.1 \times 0.9 + 0.35$

$= 1.3 \times 0.2638 + 0.09 + 0.35 = \text{Rs. } 0.78294 \text{ lakhs}$

(ii) Annual cost of the bituminous pavement

$= (4.2 - 2.0)\,CRF_{(i = 0.08,\ n = 15)} + 0.08 \times 2.0 + 0.25$

$= 2.2 \times 0.11683 + 0.16 + 0.25 = \text{Rs. } 0.66703 \text{ lakhs}$

The average annual cost of the bituminous pavement is lower and therefore works out to be more economical when compared with the pavement with WBM base course.

14.4 ECONOMIC ANALYSIS

14.4.1 Basis of Analysis

Economic analysis of a highway improvement aims at determining the monetary benefit due to the additional expenditure. The analysis also helps to decide the most economical proposal among various alternatives. In the analysis for economic justification of the proposed improvement, it is required to use judgment such as quantitative selection of the factors in which annual highway cost depends and the estimation of AADT of each class of vehicle considering the normal increase in traffic and the generated traffic.

14.4.2 Methods of Analysis

The procedure for the economic evaluation of highway projects consists of qualification for cost component and the benefits arising out of the project and to evaluate by one of the methods of analysis.

There are several methods of economic analysis. Some of the common methods are, Annual-cost Method, Rate-of-Return Method and Benefit-Cost Method.

Annual-Cost Method

The annual cost of each element of capital improvement is found by multiplying by the appropriate CRF value calculated for the assume life span. The annual cost C_r may be found using the relation : (Eq. 14.3).

$$C_r = P.\frac{i(1+i)^n}{(1+i)^n - 1} = P\,(CRF)$$

The total annual cost of an improvement is the sum of all annual costs of capital recovery (Cr) plus annual maintenance and road user costs. Table 14.1 gives the CRF values for various values of i and n obtained from equation 14.4.

Table 14.1 Capital Recovery Factor (CRF) for Various Life and Interest Rates

$$CRF = \frac{i(1+i)^n}{(1+i)^n - 1}$$

Life, n Years	Interest rate, I							
	0%	2%	3%	4%	5%	6%	8%	10%
5	0.20000	0.21216	0.21835	0.2243	0.23097	0.23740	0.25046	0.26380
10	0.10000	0.11133	0.11723	0.12329	0.12950	0.13587	0.14903	0.16275
15	0.06667	0.77083	0.08377	0.08994	0.09634	0.10296	0.11683	0.13147
20	0.05000	0.06116	0.06722	0.07358	0.08024	0.08718	0.10185	0.11746
25	0.04000	0.05122	0.05743	0.06401	0.07095	0.07823	0.09368	0.11016
30	0.03333	0.04465	0.05102	0.05783	0.06505	0.07265	0.08883	0.10608
35	0.02857	0.04000	0.04654	0.05358	0.06107	0.06897	0.08580	0.10369
40	0.02500	0.03656	0.04326	0.05052	0.05828	0.06646	0.08386	0.10226
50	0.02000	0.03182	0.03887	0.04655	0.05478	0.06344	0.08174	0.10086
60	0.01667	0.02877	0.03613	0.04420	0.05283	0.06188	0.08080	0.10033
80	0.01250	0.02516	0.03311	0.04181	0.05103	0.06057	0.08017	0.10005
100	0.01000	0.02320	0.03165	0.04081	0.05038	0.06018	0.08004	0.10001

Rate-of-Return Method

There are number of variations for the determination of rate of return of a highway improvement. In the rate of return method, the interest rate at which two alternative solutions have equal annual cost is found. If the rate of return of all proposed projects are known, the priority for the improvement could be established.

Road Research Laboratory (London) has recommended a simplified procedure of rate of return method. The percentage rate of return R is given by

$$R = \frac{O + A - M}{P} \times 100 \qquad (14.6)$$

where O = savings in annual road user costs

A = annual savings in accident costs

M = additional maintenance cost per annum

p = capital cost of improvement

Benefit cost ratio method

The principle of this method is to assess the merit of a particular scheme by comparing the annual benefits with the increase in annual cost

$$\text{Benefit cost ratio} = \frac{\text{annual benefits from improvement}}{\text{annual cost of the improvement}}$$

$$= \frac{R - R_1}{H_1 - H} \qquad (14.7)$$

where R = total annual road user cost for existing highway

R_1 = total annual road user cost for proposed highway improvement

H = total annual cost of existing road

H_1 = total annual cost of proposed highway improvement

The benefit-cost ratios are determined between alternate proposals and those plans which are not attractive are discarded. Then the benefit cost ratios for various increments of added investment are computed to arrive at the best proposal. In order to justify the proposed improvement, the ratio should be greater than 1.0. However, the choice of interest rate would affect the results of the benefit-cost solutions.

Example 14.5

It is proposed to widen a stretch of a single lane road of length 40 km to two lanes at a total cost of Rs. 6.5 lakhs per km and the rate of interest is 10% per year. The annual cost of maintenance of the existing single lane road is Rs. 7,000 per km. and that of the improved two lane road is Rs. 9,000 per km. The average vehicle operation cost on the existing road is Rs. 1.30 per vehicle-km and that on the improved is estimated to be Rs 1.15 per veh-km. If present traffic is 2000 motor vehicles per day and by the end of 15 years design period the traffic is estimated to be doubled, determine whether the investment on the improvement of the road is economically viable, during the 15 years period.

Solution

(i) Average traffic during the design period = (2000 + 4000)/2 = 3000 mv/day

Average road user cost on existing road per year = 365 × 40 × 3000 × 1.30 = Rs. 569.4 lakhs

(ii) Average road user cost on improved road per year = 365 × 40 × 3000 × 1.15 = Rs. 503.7 lakhs

Total benefits = 569.4 – 503.7 = Rs. 65.7 lakhs

Total cost of improvement, P = 6.5 × 40 = Rs. 260.0 lakhs

CRF (for n = 15 and i = 10%) = 0.13147

Present annual cost of improvement, C_r = P. CRF

= 260 × 0.13147 = Rs. 34.182 lakhs

Additional maintenance cost per year = Rs. (9000 – 7000) × 40 = Rs. 0.80 lakhs

(iii) Total cost = present annual cost of improvement and additional maintenance cost per year = 34.182 + 0.80 = Rs. 34.982 lakhs

(iv) Benefit cost ratio = $\dfrac{65.7}{34.982}$ = 1.878

As this is greater than 1.0, the project is economically viable.

Example 14.6

An existing road link of length 21 km connecting a tourist centre has a bad alignment, inadequate width and poor surface condition. Proposal A is to increase pavement width and re-surface the road on the existing road. Two other alternatives B and C have been proposed along new alignment, their road lengths being 17.5 and 16.7 km respectively.

The present traffic is 400 passenger cars per day with negligible commercial vehicles. The traffic is expected to be doubled by the end of the 60 year period. The average speed of vehicles on alignment A is 40 kmph and that on alignments B and C is 75 kmph. Use charts for vehicles Running Cost (Fig. 14.6) and time cost Rs. 4.50 per vehicle-hour curve (Fig. 14.3) for estimating the vehicle operation cost.

The estimated details of cost and life for the three proposals are given below. Assuming the rate of interest as 8% and annual maintenance cost Rs. 5,000 per km, analyse the economics by (i) annual cost method and (ii) Benefit cost ratio method.

Element	Estimated useful life years	Cost in thousand rupees		
		Proposal A	Proposal B	Proposal C
Right of way	100	0	270	310
Grading	50	150	290	330
Structures	50	160	250	290
Pavement	10	310	1550	1450

Solution

(i) Annual Cost Method

The annual cost of each element may be found by multiplying first cost P by the appropriate CRF value. The CRF value corresponding to the specified rate of interest i and number of interest periods n may either be taken from the Table 14.1 or may be calculated using Eq. 14.4. As an example, the annual cost of right of way of Proposal B with P = 270,000, n = 100 yrs. and i = 8% may be calculated, as follows

From Table 14.1, CRF = 0.08004;

or From Eq. 14.1. CRF = $\dfrac{i(1+i)^n}{(1+i)^n - 1} = \dfrac{0.08(1+0.08)^{100}}{(1+0.08)^{100} - 1}$

= 0.08004 or say, CRF = 0.08

Therefore annual cost C_r = 270,000 × 0.08 = Rs 21,600

The details of the highway cost and the road user cost have been worked out for three proposals in the Table 14.2.

Table 14.2 Solution of Example 14.5

1. Highway Cost

Cost elements	Proposal A	Proposal B	Proposal C
(a) Annual cost of right of way at i = 8%, n = 100 yrs. CRF = 0.08	Rs. 0	27,000 × 0.08 = Rs. 21,600	310,000 × 0.08 = Rs. 24,800
(b) Annual cost gradient at i = 8%, n = 50 yrs. CRF = 0.0817	150,000 × 0.0817 = Rs. 12,255	290,000 × 0.0817 = Rs. 23,700	330,000 × 0.0817 = Rs. 27,000
(c) Annual cost of structures at i = 8%, n = 50 yrs. CRF = 0.0817	160,000 × 0.0817 = Rs. 13,072	250,000 × 0.0817 = Rs. 20,400	290,000 × 0.0817 = Rs. 23,800
(d) Annual cost of pavement at i = 8%, n = 10 yrs. CRF = 0.149	310,000 × 0.149 = Rs. 46,200	1550,000 × 0.149 = Rs. 228,700	1450,000 × 0.149 = Rs. 216,600
(e) Annual maintenance cost	21 × 5000 = Rs. 105,000	17.5 × 5000 = Rs. 87,500	16.7 × 5000 = Rs. 83,500
1. Total annual highway cost, H	H_A = 176,527	H_B = 381,900	H_C = 375,100

2. Road User Cost

Cost elements	Proposal A	Proposal B	Proposal C
Average traffic during design life (400 + 800)/2 = 600 vehicles/day Total vehicle operation cost Rs/Veh. km. At speed of (Fig. 14.6)	60 kmph, Rs. 0.55	75 kmph, Rs. 0.56	75 kmph, Rs 0.56
(f) Annual running cost for the road length	0.55 × 600 × 365 × 21 = 2529,450	0.56 × 600 × 365 × 17.5 = 2146,200	0.56 × 600 × 365 × 16.7 = 2048,088
Time cost per veh. km. at speed of (Rs. 4.50 curve of Fig. 14.3)	60 kmph, Rs. 0.12	75 kmph, Rs. 0.09	75 kmph, Rs 0.09
(g) Time cost of all the vehicles for the road length	0.12 × 600 × 365 × 21 = 551,880	0.09 × 600 × 365 × 17.5 = 344,925	0.09 × 600 × 365 × 16.7 = [illegible]
2. Total annual road user Cost, R	R_A = 3081,330	R_B = 2491,125	[illegible]
3. Grand total annual cost of proposal = sum of total annual highway cost and road user cost = 1 + 2	3257,857	2873,025	2752,345

From the bottom line of this Table it may be seen that the grand total of the Annual Cost values for the Proposals A, B and C are Rs. 3257,857, Rs. 2873,025 and Rs. 2752,345 respectively. Therefore, proposals C with the lowest value of total annual cost is the best or most economical proposal.

(ii) Benefit-cost method

From Table of Annual Cost : (Table 14.2)

Total Annual highway cost of proposal A = H_A = Rs. 176,527

Total annual road user cost for proposal A = R_A = 3081,330

Total annual highway cost of proposal B = H_B = Rs. 381,900

Total annual road user cost for proposal B = R_B = Rs. 2491,125

Benefit-cost ratio,

$$\frac{B}{A} = \frac{R_A - R_B}{H_B - H_A} = \frac{3081{,}330 - 2491{,}125}{381{,}900 - 176{,}527} = \frac{590{,}205}{205{,}373} = 2.874$$

Total annual highway cost of proposal C = H_C = Rs. 3,75,100

Total annual road user cost proposal C = R_C = Rs. 2377,245

Benefit – cost ratio,

$$\frac{C}{A} = \frac{R_A - R_C}{H_C - H_A} = \frac{3081{,}330 - 2377{,}245}{375{,}100 - 176{,}527} = \frac{704{,}085}{198{,}573} = 3{,}546$$

Therefore, alternative C is the best one with higher benefit-cost ratio.

14.5 HIGHWAY FINANCE

Basic principle in highway financing is that the funds spent on highways are recovered from the road users. The recovery may be both direct and indirect.

Two general methods of highway financing are

Pay-as-you-go method

Credit financing method

In *pay-as-you-go* method, the payment for highway improvements, maintenance and operation is made from the central revenue. In *credit financing* method, the payment for highway improvement is made from borrowed money and this amount and the interests are re-paid from the future income.

Distribution of highway cost

The question of distributing highway cost among the Government, road-user and other has been a disputed task in several countries. Many economists are of the view that the financial responsibility for roads should be assigned only among the beneficiaries on the basis of the benefit each one receives.

There are several theories suggesting the method of distribution of highway taxes between passenger cars and other commercial vehicles like the trucks. However in India the annual revenue from transport has been much higher than the expenditure on road development and maintenance. Therefore there is no problem of distributing the highway cost among other agencies. Also the taxation on vehicles is being considered separately by the states and there seems to be no theory followed for the distribution of taxes between various classes of vehicles.

Sources of Revenue

The various sources from which funds necessary for highway development and maintenance may be made available, are listed below :

Taxes on motor fuel and lubricants

Duties and taxes on new vehicles and spare parts including tyres

Vehicles registration tax

Special taxes on commercial vehicles

Other road user taxes

Property taxes

Toll taxes

Other funds set apart for highways

There should be an equitable distribution of revenues available for highways.

Highway financing in India

The responsibility of financing different roads lies with the Central Government, State Governments and local bodies including Corporations, Municipalities, District Boards and Panchayats.

Taxes levied by Central Government for highway financing are :

Duties and taxes on motor fuel

Excise duty on vehicles and spare parts, tyre etc.

Excise duty on oils, grease, etc

Taxes levied by the State Governments include :

Registration fees for vehicles and road tax

Permits for transport vehicles

Passenger tax on buses

Sales tax on vehicle parts tyre etc.

Fees on driving licenses

Taxes levied by local bodies are mainly the toll tax.

Ever since the introduction of Central Road Fund (CRF) in the year 1929 by taxing motor fuel, this has been the main source of finance for the State Government to meet the road development needs, without having to go through the time consuming process of special sanctions each time. However of late the CRF is also being merged with the general revenue. In March 1976 the Lok Sabha has passed the resolution of the Ministry of Transport ensuring the existence of the CRF separately with the specified objectives. An amount of not less than 3.5 paise per litre out of the duty of customs and excise on motor spirit would be set apart towards the CRF for the road development. While utilising this fund, greater attention would be given to schemes of all-India importance. Twenty percent of the fund would be retained by the central Government as reserve. The fund will also be used for road research schemes, traffic studies, economic surveys and training arrangements for young engineers. The gross revenue from road transport in India during the sixth plan period 1978-83, 1980-85 was about Rupees 12,000 crores.

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PROBLEMS

1. Discuss the importance of highway economy studies.
2. Explain briefly various factors affecting the vehicle operation cost.
3. How is the cost of highways analysed ?
4. Write a note on economics of highway pavements.
5. Explain benefit-cost analysis and its significance.
6. Write a note on Highway financing.
7. Calculate the operation cost of passenger car for 150 km length of a rural highway with no sharp curves for most economical vehicle operation using the following data and charts given in Fig. 14.1, 14.2, 14.3 and 14.4

   Gradients: level for 30 km, 0 to 2% for 60 km, 4 percent for 40 km and 6% for 20 km.

   Pavement surface condition : good

   Fuel cost : Rs. 8.80 per litre

   Tyre cost : Rs. 910 per tyre

   No. of stops enroute : 10 stops without delay 5 with 45 secs. delay and 4 with 60 secs. delay.

   Time cost per vehicle-hour = Rs. 3.00

   Depreciation cost of vehicle = 5 paise per km

8. The distance between two towns is 200 km. Calculate the average operation cost per vehicle-km for a desired average running speed of 60 kmph from the following data :

   Fuel cost = Rs. 8.90 per litre

   Tyre cost = Rs. 900 per tyre

   Gradients = 80 km level, 80 km 0-3% grade, and 40 km 3-5 percent grade.

   Pavement surface condition = good.

   Speed reduction to 30 kmph at curves and to 20 kmph at 8 congested zone.

   Total 12 stops with average 60 sec. delay and time cost per vehicle-hour = 4.50

   Depreciation cost of vehicle = 6 paise per km.

   Figures 14.1 to 14.5 may be used.

9. The details of three alternate proposals for strengthening of a highway pavement are given below : Determine which one is more economical, if its rate of interest payable in all the cases is 10% per annum and there are average 3500 motor vehicles per day, with annual growth rate of 5.5%.

S. No.	Overlay type	Design life, years	Construction cost, Rs. (lakhs) per km	Average maintenance cost, Rs. per km (during design life)	Vehicle operation cost, Rs. per veh-km (during design life)
(i)	WBM + PMC	5	4.5	15,000	1.52
(ii)	BM + PMC	8	6.8	8,000	1.45
(iii)	BM + AC	12	9.2	4,000	1.35

10. An existing road link of length 21 km connecting a tourist centre has a bad alignment, inadequate width and poor surface condition. Proposal X is to increase width and resurface pavement on the existing road. Two other alternatives Y and Z have been proposed along new alignment, their road lengths being 17.5 and 16.7 km respectively.

    The present traffic is 500 passenger cars per day with negligible commercial vehicles. The traffic is expected to be doubled by the end of the 10 year period. The average speed of vehicles on alignment X is 45 kmph and that on alignments Y and Z is 60 kmph. Use charts for vehicle Running Cost and Time Cost per vehicle hour curve. These are given in Fig. 14.3 and 14.6.

    The estimated details of cost and life for the three proposals are given below. Assuming the rate of interest as 8% and annual maintenance cost Rs. 7000 per km, analyse the economics by (i) Annual cost method and (ii) Benefit cost ratio method.

Element	Estimated useful life, years	Cost in thousand rupees		
		Proposal X	Proposal Y	Proposal Z
Right of way	100	0	270	310
Grading	50	150	290	330
Structures	50	160	250	290
Pavement	10	310	1550	1450

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