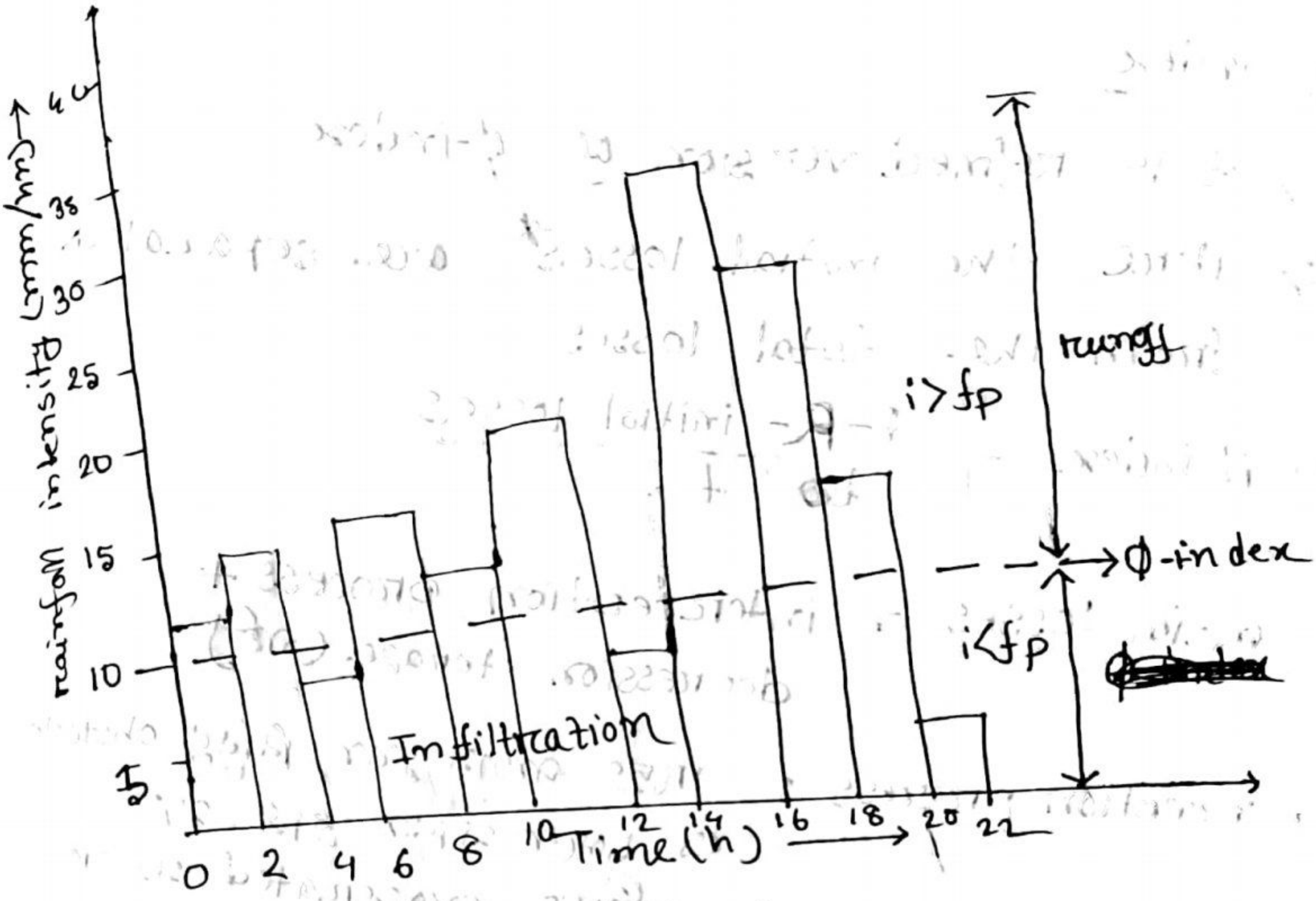

Hyetograph

<u>φ index</u> : The φ-index is the average rainfall above which the rainfall volume (intensity) is equal to the runoff volume (intensity)

$$\phi \text{ index} = \frac{P_e - R}{t_e}$$

P_e = excess rainfall ($i > f_p$)

t_e = time of excess rainfall

R = total runoff

P = all kind of rainfall ($i > f_p$ or $i < f_p$)

* মোটকথায় φ-index হলো soil এর maximum infiltration capacity. তাই $i < f_p$ থাকবে হিসেবে ধরা হবে।

W- index

⇒ it is refined version of ϕ-index

⇒ Here the initial losses* are separated from the total losses

⇒ $$\text{W-index} = \frac{P - R - \text{initial losses}}{t}$$

* Initial losses = interception process + depression storage (গর্ত).

<u>interception process</u> : গাছের পাতায়, বা বিভিন্ন obstacle এ জমে থাকা বৃষ্টি যা পরবর্তীতে evaporated হয়ে যায়।

⇒ W-index সবসময় ϕ-index থেকে ছোট হবে। কারণ এখানে initial loss ~~কে~~ count করা হয় না।

⇒ W-index is more accurate than the ϕ-index

*** In estimating the max^m^ floods for design purposes, in the absense of any other data, <u>ϕ-index</u> value is taken as <u>0.10cm/hr</u>

Calculation of Runoff

$$R = \sum_{i}^{n} (I_i - \phi)\Delta t$$

Δt = time interval

I_i is always greater than ϕ in that case

only $I_i > \phi$ is acceptable in the case

Math Problems :

① The rate of rainfall for the successive 30min period of a 3hr storm are: 1.6, 3.6, 5.0, 2.8, 2.2, 1.0 cm/hr. The surface runoff is estimated to be 3.6cm. Determine ϕ-index & W-index.
[BIWTA'19, BWDB 18]

Sol^n:

$$P = (1.6 + 3.6 + 5.0 + 2.8 + 2.2 + 1.0) \times \frac{30}{60}$$

$$= 8.1 \text{ cm}$$

$$\therefore W = \frac{P - R - \text{initial loss}}{t_r}$$

$$= \frac{8.1 - 3.6 - 0}{3}$$

$$= 1.5 \text{ cm/hr}$$

$\because$ ϕ-index > W-index এর ক্ষেত্রে Pe বের করার জন্য 1.5 cm/hr এর চেয়ে কম intensity এর value গুলো বাদ দিতে হবে।

$$Pe = 8.1 - 1.0 \times \frac{30}{60}$$

$$= 7.6\,cm$$

$$\therefore \phi \text{ index} = \frac{Pe - R}{\Sigma te}$$

$$= \frac{7.6 - 3.6}{2.5} \quad [30 \text{ minute বাদে}]$$

$$= 1.6\ cm/hr$$

(2) The intensity/rate of rainfall for successive 20min period of a 140min storm 2.5, 2.5, 10.0, 7.5, 1.25, 1.25, 5 cm/hr. Assume ϕ index = 3.2cm/hr. Find net runoff, total rainfall & W-index if inital loss is 0.96cm.

Soln:

(i) Total rainfall, $P = (2.5 + 2.5 + 10 + 7.5 + 1.25 + 1.25 + 5) \times \frac{20}{60}$ cm

$$= 10\,cm$$

(ii) Runoff, $R = \sum_{i=1}^{n} (i - \phi)\Delta t$

$$= [(10 - 3.2) + (7.5 - 3.2) + (5 - 3.2)] \times \frac{20}{60}$$

$$= 4.3\,cm$$

(ii) W index $= \dfrac{P - R - \text{initial loss}}{t}$

$$= \frac{10 - 4.3 - 0.06}{\frac{140}{60}}$$

$$= 2.42 \text{ cm/hr}$$

(3) Find the ϕ-index if the direct runoff is 4.9 cm [BWDB-19]

Time (hr)	1	2	3	4	5
P (cm)	1.5	2.5	4.1	3	1.9

Soln: যদি প্রশ্নের অনুসারে ϕ-index বের করতে বলে হয়। • w-index
কিন্তু সাধারণত অনুসারে W-index বের করতে হয় কারণ $\phi >$ W index
থেকে ϕ-index বের করতে হয়

$P = 1.5 + 2.5 + 4.1 + 3 + 1.9 = 13$ cm

W index $= \dfrac{13 - 4.9}{5} = 1.62$ cm/hr

Now, $P_e = 13 - 1.5 = 11.5$ cm [hr এ দেয়া আছে তাই minute নিয়ে সমস্যা নাই]

$$\therefore \phi \text{ index} = \frac{P_e - R}{t_e} = \frac{11.5 - 4.9}{4} = 1.65 \text{ cm/hr}$$

(4) A 6h storm with hourly intensities of 7, 18, 25, 12, 10, 3 mm/h produces a runoff of 33 mm. Then ϕ-index is ?

Soln:

$$W\text{-index} = \frac{75-33}{6} = 7\ \text{mm/h}$$

$$\phi\text{-index} = \frac{\cancel{(75-7)-33}}{\cancel{5}} = \frac{(75-7-3)-33}{4}$$

$$= 8\ \text{mm/h}\ \underline{\text{Ans}}$$

(5) A 4h storm had 4cm of rainfall & the resulting direct run-off was 2cm. If the ϕ-index remains at the same value, the runoff due to 10cm of rainfall in 8h in the catchment is what?

Soln: ϕ index $= \frac{4-2}{4} = 0.5$

again, $0.5 = \frac{10-R}{8}$

$\Rightarrow R = 6\text{cm}$ Ans

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(6) A storm with 10 cm of precipitation produced a direct runoff of 5.8 cm. The duration of the rainfall was 16 hrs and its time distribution is given below. Estimate ϕ-index

Time (hr)	0	2	4	6	8	10	12	14	16
Cumulative rainfall (cm)	0	0.4	1.3	2.8	5.1	6.9	8.5	9.5	10
increment (cm)	0	0.4	0.9	1.5	2.3	1.8	1.6	1.0	0.5
intensity (cm/hr)	0	0.2	0.45	0.75	1.15	0.9	0.8	0.505	

Soln:

$$W\text{-index} = \frac{10 - 5.8}{16} = 0.2625 \text{ cm/hr}$$

$$\phi \text{ index} = \frac{(10 - 0.4 - 0.5) - 5.8}{12} = 0.275 \text{ cm/hr}$$

duration of rainfall excess = 12 hr

Date: / /

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The intensity/rate of rainfall for successive 20min period of a 140min storm 2.5, 2.5, 10.0, 7.5, 1.25, 1.25, 5cm/hr. Assume ϕ-index = 3.2cm/hr. Find net runoff, total rainfall & W-index if initial loss is 0.06cm..

Solⁿ:

(i) Total rainfall, $P = (2.5 + 2.5 + 10 + 7.5 + 1.25 + 1.25 + 5) \times \frac{20}{60}$ cm

$= 10$ cm

(ii) Runoff, $R = [(10 - 3.2) + (7.5 - 3.2) + (5 - 3.2)] \times \frac{20}{60}$

$= 4.3$ cm

(iii) W index $= \frac{P - R - \text{initial loss}}{t}$

$= \frac{10 - 4.3 - 0.06}{140/60}$

$= 2.42$ cm/hr

Date : / /

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(2) A 8 hours storm produced a surface runoff of 11.6 cm. Estimate the infiltration-index of the storm. [DMTCL Line 55]

Storm time(h)	1	2	3	4	5	6	7	8
Rainfall intensity (cm/h)	0.8	1.8	3.0	4.6	3.6	3.2	2.0	1.0

Solution:

Total precipitation = 0.8+1.8+3.0+4.6+3.6+3.2+2.0+1.0 = 20 cm

$$W\text{-index} = \frac{20-11.6}{8} = 1.05\ \text{cm/hr}$$

1hr এবং 8hr storm intensity W-index থেকে ছোট, তাই এদের জন্য infiltration হবেনা।

$$\therefore \text{infiltration index} = \frac{(20-0.8-1.0)-11.6}{8-2}$$

$$= 1.1\ \text{cm/hr} \quad \underline{A}$$

এর মানে প্রথম এবং 8th hour এ কোন infiltration হবেনা। 2nd থেকে 7th বা 6 ঘণ্টায় গড় infiltration 1.1 cm/hr

Date :

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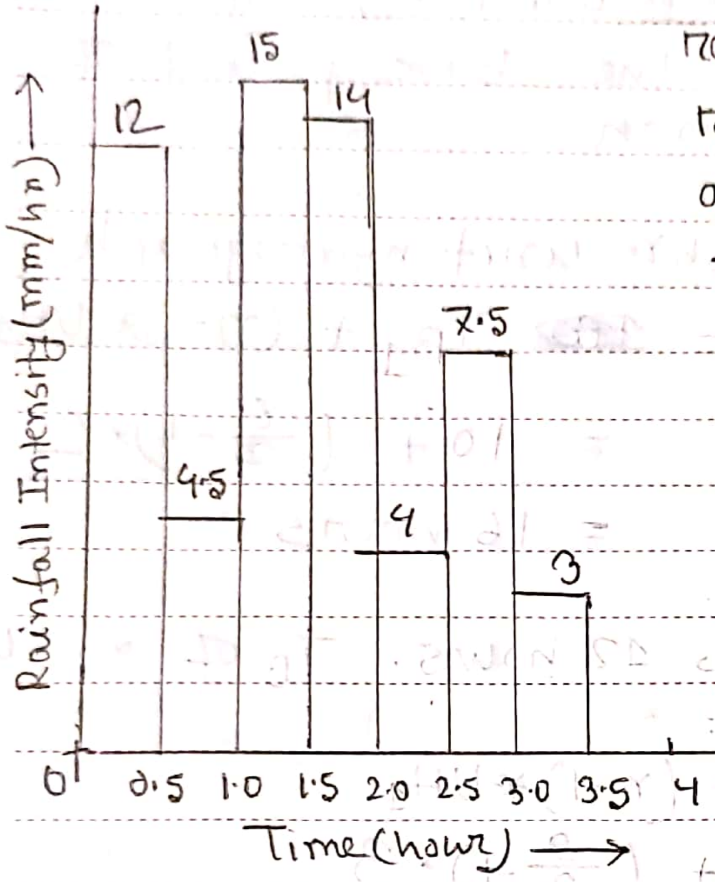
* Rainfall intensity with respect to time is given below:



Rainfall depth is 3cm. If the rainfall event has produced a direct runoff of 1.6cm, calculate the ϕ-index of the event.

Sol^n: W-index $= \dfrac{3 - 1.6}{3.5} = 0.4\,cm/hr$

$= 4\,mm/hr$

mm [illegible]

$\therefore$ ϕ-index $= \dfrac{\left(30 - 4 \times \frac{T}{2} - \frac{3}{2}\right) - 16}{3.5 - 1}$ → mm

$= 4.2\,mm/hr$

[illegible] discharge = $1.2\,m^3/s$

[illegible] base flow = $1.2\,m^3/s$

Time (min)	Intensity (mm/h)	Discharge (m^3/s)
	—	1.2
0		1.2
30	8 — initial loss	3.9
60	18	8.5
90	35	6
120	25	4.5
150	8	3.1
180	16	1.2
210	22	

Catchment area is $1.5\,km^2$. Assume the rainfall amount in the first half-an-hour is the initial loss & the baseflow is $1.2\,m^3/sec$. determine ϕ-index ~~ϕ-index~~

(a) ~~[illegible]~~

Solⁿ: $P = \frac{30}{60}[8+18+35+25+8+16+22] = 66\,mm$

Runoff volume $= \left[(1.2-1.2)+(3.9-1.2)+(8.5-1.2)+(6-1.2)+(4.5-1.2)+(3.1-1.2)+(1.2-1.2)\right] \times 60 \times 30$

$\boxed{m^3/s = 60\,m^3/m = 60 \times 30\,m^3 \text{ per } 30\text{ min}}$

$= 36000\,m^3$

Runoff depth $= \frac{36000\,m^3}{1.5 \times 10^6\,m^2} = 0.024\,m = 24\,mm$

$\frac{210-30}{60} = 3\,hr$

$\therefore$ W index $= \frac{P-R-L}{t} = \frac{66-24-4}{3} = 12.67\,mm/hr$

First 30 min intensity $= 8\,mm/hr$ ∴ [illegible] 30 min ∴ [illegible] $= 4\,mm$

$\therefore P_e = (18 + 35 + 25 + 16 + 22) \times \frac{30}{60} = 58$

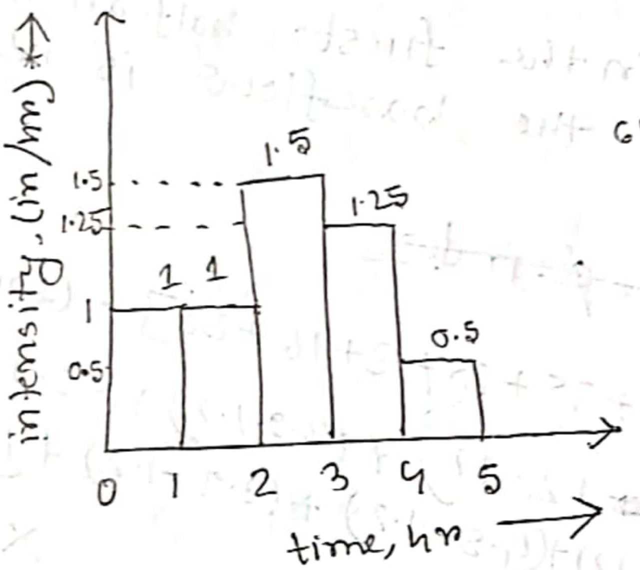
~~te = 3.5 60~~ $t_e = 3$ hrs (30 min, 30min করে 1 ঘণ্টা ধরি)

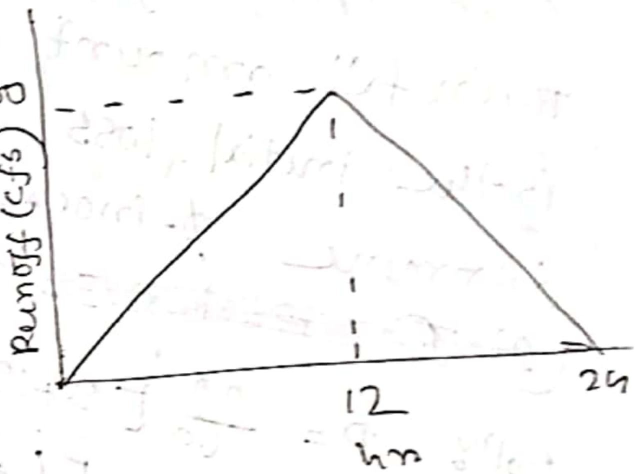
$\therefore \phi \text{ index} = \frac{58 - 24}{2.5} = 13.6$ mm/hrs

***

Intensity vs time ⇒ Hyetograph

Flow (Runoff) vs time ⇒ Hydrograph





Given, ϕ index = 0.5 in/hr

Determine total volume of runoff & the watershed area.

Soln:

$P = 1 + 1 + 1.5 + 1.25 + 0.5 = 5.25$ in

Runoff depth, $R = (1 - 0.5) + (1 - 0.5) + (1.5 - 0.5) + (1.25 - 0.5) + (0.5 - 0.5)$

$= 2.75$ in

Wrong

ϕ. $\phi\text{-index} = \dfrac{P_e - R}{t_e}$

$\Rightarrow 0.5 = \dfrac{5.25 - R}{5} \Rightarrow R = 5.25 - 2.5 = 2.75$

$\text{Runoff volume} = \dfrac{1}{2} \times \overset{600 \times 3600}{\cancel{}} \times 24 = \cancel{}\ 25920000\ ft^3$

$\therefore \dfrac{2.75}{12} ft = \dfrac{\cancel{}\,25920000}{\text{Area}}$

$\Rightarrow \text{Area} = \dfrac{25920000 \times 12}{2.75} = 113105454.5\ ft^2$

Hydrograph

DRH = Direct Runoff hydrograph
FH = Flood hydrograph

DRH = FH − base flow

$$\text{* Unit hydrograph (UH)} = \frac{\text{DRH}}{\text{rainfall access (cm)}}$$

runoff

$$\text{* Peak flow of UH} = \frac{\text{Peak of DRH}}{\text{rainfall access (cm)}}$$

* Runoff volume = Catchment area * runoff depth

* UH का 1cm या 1in (unit) runoff द्वारा DRH

Flood hydrograph = DRH + baseflow
= UH* runoff + baseflow

* The peak of flood hydrograph due to a 3h duration isolated storm in a catchment is 270 m³/sec. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss is 0.3 cm/hr and a constant ~~base fol~~ baseflow of 20 m³/s. Estimate the peak of the 3-hr unit hydraugraph of this catchment.

Soln: Peak of DRH $= 270 - 20 = 250\ m^3/s$

~~Peak of UH = 250/50 =~~

runoff, $R = P - \phi t$
$= 5.9 - 0.3 \times 3$
$= 5\ cm$

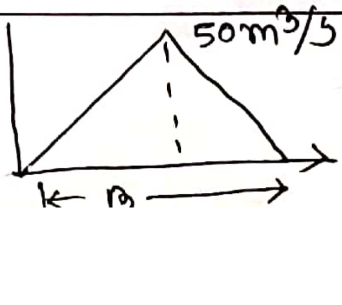
$\therefore$ Peak of UH $= \frac{250}{5} = 50\ m^3/s$ Ans

* From previous questions, If the area of catchment is 567 km², Determine base width of a 3hr UH by assuming it to be a triangular shape.

Soln: Area of UH = catchment area × 0.01 (1 cm)

$\Rightarrow \frac{1}{2} \times B \times 50 \times 60 \times 60 = 567 \times 10^6 \times 0.01$

$\therefore B = 63$ hours Ans



50 m³/s
$= 50 \times 3600\ m^3/hr$

** A unit hydrograph for a 200-acre watershed has a triangular shape and a duration of 4 hrs. What is the max^m flow rate (cfs) in the unit hydrograph?

$A = 200 \text{ acres} = 200 \times 43560 = 8712000 \text{ ft}^2$

$\because$ unit hydrograph $= 1 \text{ inch} = \frac{1}{12} \text{ ft}$

$\therefore$ Runoff, $R = $

$$R = \frac{\frac{1}{2} \times 4 \times Q}{8712000}$$

$$\Rightarrow \frac{1}{12} = \frac{2Q}{8712000}$$

$$\Rightarrow Q = 363000 \text{ ft}^3/\text{hr}$$

$$= 100.83 \text{ ft}^3/\text{sec (cfs)}$$

* A flood hydrograph of a river draining a catchment of 189 km² due to a 6h isolated storm is in the form of a triangle with a base of 66h and a peak ordinate of 30 m³/sec occurring at 10 hours from start. Assume zero base flow. Develop 6-hr unit hydrograph for this catchment.

<u>Solⁿ:</u> Runoff volume $= \frac{1}{2} \times 66^{h} \times (30 \times 60 \times 60)$ $30\,m^3/h$

$= 3564000\,m^3$

Runoff depth $= \dfrac{~~3564~~\ 3564000}{189 \times 10^6}$

$= 0.01885\,m$

$= 1.885\,cm$

Peak of 6hr Unit hydrograph $= \dfrac{30}{1.885}$

$= 15.909\,m^3$

Ans

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* A 12hr unit hydrograph of a catchment is of a triangular shape with a base width of 144 hour and a peak discharge of 23 m^3/s. Calculate the catchment area in km^2.

Solⁿ:

$$\frac{1}{2} \times \overset{hr}{144} \times \underbrace{23 \times 60 \times 60}_{m^3/s \text{ to } m^3/hr} = \text{Area} \times 0.01$$

1 cm to m

$$\Rightarrow \text{Area} = 59616000\ m^2$$

$$= \cancel{59.616\ km^2}$$

$$= 596.16\ km^2$$

12hr এর কোন ভূমিকা নাই

1m = 0.01 cm

Math Problems

DWASA - 17

(1) Rainfall of magnitude 3.8cm & 2.8cm occuring on two consecutive 4-h durations on a catchment of area 27 km^2 produced the following hydrograph of flow at the outlet of the catchment. Estimate the rainfall excess and ϕ-index.

Flood hydrograph

Time (hr)	-6	0	6	12	18	24	30	36	42	48	54	60	66
flow (m^3/s)	6	5	13	26	21	16	12	9	7	5	5	4.5	4.5

Soln: base flow = $5 m^3/s$

[Flood hydrograph থেকে base flow বিয়োগ করলে DRH পাওয়া যায়]

Time (hr) →	0	6	12	18	24	30	36	42	48
direct flow (m^3/s) →	0	8	21	16	11	7	4	2	0

[শূন্য আসে zero আসবে negative হবে না]

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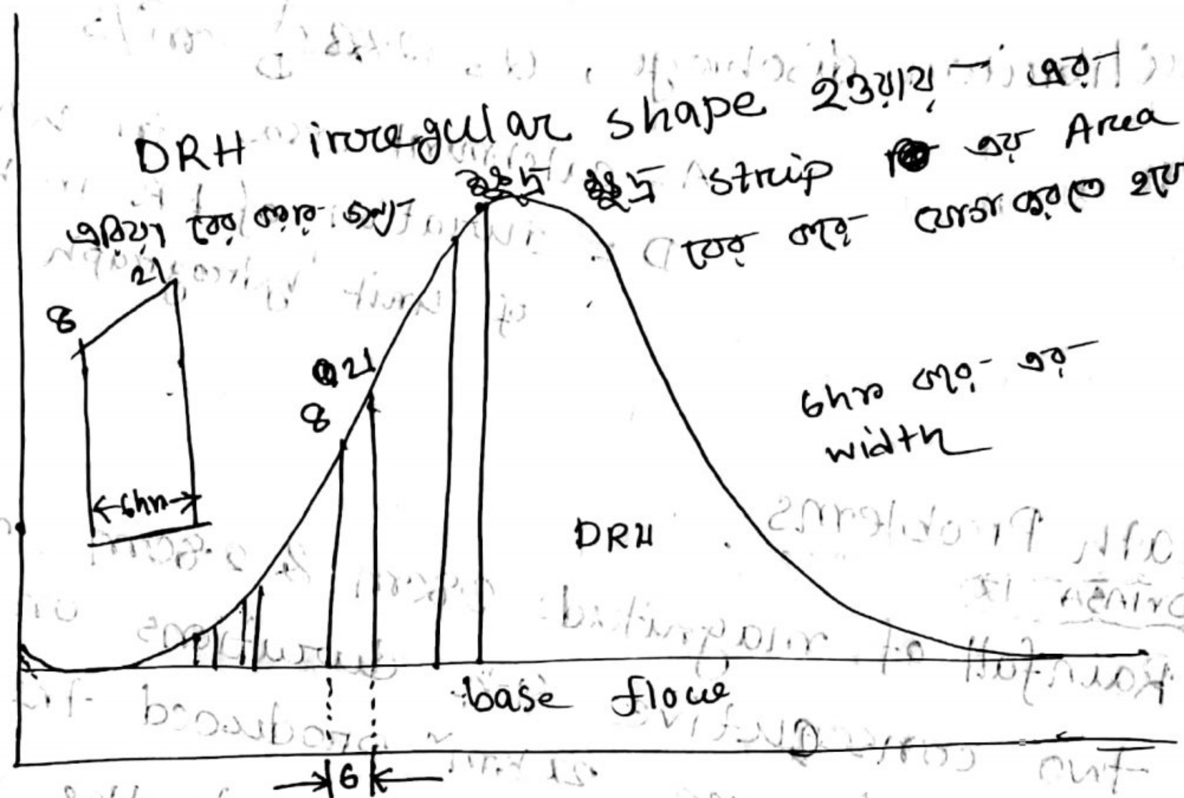
Runoff volume = Area of DRH



$$\text{Runoff volume} = (6\times60\times60)\times\Big[\tfrac{1}{2}(0+8)+\tfrac{1}{2}(8+21)+\tfrac{1}{2}(21+16)+\tfrac{1}{2}(16+11)+\tfrac{1}{2}(11+7)+\tfrac{1}{2}(7+4)+\tfrac{1}{2}(4+2)+\tfrac{1}{2}(2+0)\Big]$$

$$= 1.4904\times10^{6}\ m^{3}$$

$$\text{Runoff depth} = \frac{\text{Runoff volume}}{\text{catchment area}}$$

$$= \frac{1.4904\times10^{6}}{27\times10^{6}} = 0.0552\,m = 5.52\ cm$$

$\therefore$ rainfall excess = 5.52 cm

Total rainfall = 3.8 + 2.8 = 6.6 cm

Duration = 4+4 = 8 hrs

ϕ index = $\frac{6.6 - 5.52}{8}$ = 0.135 cm/hr Ans

(WRE 15)

(2) The peak of flood hydrograph due to a 3h duration isolated strom in a catchment is 270 m³/s. The total depth of rainfall is 5.9 cm. Assuming an average infiltration loss of 0.3 cm/h & a constant base flow of 20 m³/s, estimate the peak of the 3h unit hydrograph of this catchment. If the area of the catchment is 567 km² determine the base width of the 3h unit hydrograph.

Soln: Peak of DRH = (270 − 20) m³/s (Total − base)

= 250 m³/s

runoff depth = 5.9 cm − 0.3 cm/h * 3h (infiltration loss)

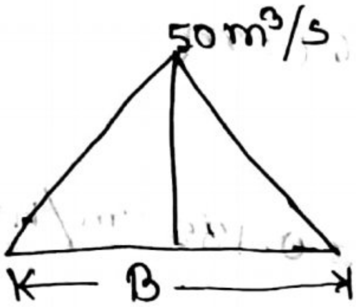
= (5.9 − 0.9) cm

= 5 cm

Peak of unit hydrograph = $\frac{250}{5}$ [∵ UH due 1 cm runoff]

= 50 m³/s

(b)



* Unit hydrograph কে সবসময় triangular shape হিসেবে assume করতে হয়।

Area of UH = Volume of 1cm runoff = Area of catchment × depth

$\Rightarrow \frac{1}{2} \times B \times 50 \times 3600 = 567 \times 10^6 \times \frac{1}{100}$

$= 5670000$

$\Rightarrow 90,000\,B$

$\Rightarrow B = 63$ hr <u>Ans</u>

graph এ width মানে hr এ সময়। কেননা আমরা ওখানে $50 m^3/s$ কে m^3/hr এ নেয়া হয়েছে।

Raghunath (আমার উত্তর [illegible] rainfall হবে)

(5) The successive three hourly ordinates of a 6hr UH for a particular basin are 0, 15, 36, 30, 17.5, 8.5, 3, 0 ~~cumec~~ cumec respectively. The flood peak observed due to a 6hr storm was 150 cumec. Assuming a constant base flow of 6 cumec & an average storm loss of 6mm/hr, determine the depth of storm rainfall and the stream flow at successive 3hr interval.

Soln: DRH Peak = flood peak − base flow

$= (150 - 6)$ cumec

$= 144$ cumec

$$\text{Runoff} = \frac{\text{DRH Peak}}{\text{UH peak}} = \frac{144}{36} = 4\text{cm}$$

$$\boxed{\because \text{ UH peak} = \frac{\text{DRH peak}}{\text{Runoff}}}$$

depth of storm rainfall = Runoff + loss

$= 4 + 6\text{mm/hr} * 6\text{hr}$

$= (4 + 0.6 * 6)$

$= 7.6\text{cm}$

Second part:

Streamflow (অথবা Flood Hydrograph) at successive 3hr interval =

প্রশ্নে UH এর ordinate দেয়া আছে। তার সাথে Runoff গুণ করলে পাবো DRH এবং DRH এর সাথে baseflow যোগ করলে পাবো Streamflow or flood hydrograph.

Now, streamflow at successive 3hr intervals are $0\times4+6=6$, $15\times4+6=66$, 150, 126, 76, 40, 18, 6 cumec respectively.

(b) BUET The hourly ordinate of 6h unit hydrograph for a particular basin is 36 cumec. The flood peak due to 6h storm was 130 cumec & The constant base flow is 6 cumec & avg. loss is 6mm/hr. Determine depth of storm rainfall & the streamflow at successive 3hr interval.

Solⁿ: (i) Peak of 6h UH = $\frac{\text{Peak of DRH}}{\text{rainfall excess}}$

$\Rightarrow$ rainfall excess = $\frac{(130-6)}{36} = 3.44$ cm

storm rainfall depth = $3.44 + 0.6\times6 = 7.04$ cm Ans

(ii) streamflow = $36\times3.44+6 = 129.84\ m^3/sec$

* The ordinate of a 2 hr unit hydrograph for a catchment are given as:

Time(h)	0	1	2	3	4
ordinate(m^3/s)	0	5	12	25	41

Calculate The ordinate (in m^3/s) of a 4 hr unit hydrograph for this catchment at the time of 3h.

Solⁿ:

4hr ব্যবধানের জন্য

Time (hr)	Ordinate of 2hr UH	Ordinate of 2hr UH lagged by 2hr	Ordinate of 4hr DRH	4hr UH $= \frac{4hr\ DRH}{2}$
0	0	–	0	0
1	5	–	5	2.5
2	12	0	12	6
3	25	5	30	15
4	41	12	53	26.5
		25	25	12.5
		41	41	20.5

∴ Ordinate of 4hr UH at 3hr duration is $15 m^3/s$

2 ঘণ্টার UH থেকে 4 ঘণ্টার UH বানাতে হবে। $\therefore n = \frac{4}{2} = 2$

$\therefore 4hr\ UH = \frac{4hr\ DRH}{n}$

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* The ordinate of a 1hr unit hydrograph at Sixty minutes interval are 0, 3, 12, 8, 6, 3, & 0 m^3/s. A two hour storm of 4cm excess rainfall occured in a basin from 10AM. considering constant base flow of $20 m^3/S$, calculate the flow of the river (in m^3/s) at 1PM.

Solⁿ:

Time always zero থেকে শুরু করতে হবে।

$2 \div 1 = 2$

	Time (hr)	1hr UH	UH of 1hr logged by 1hr	2hr DRH	2hr UH $= \frac{2hr\,DRH}{2}$
10AM	0	0	—	0	0
11AM	1	3	0	3	1.5
12pm	2	12	3	15	7.5
1PM	3	8	12	20	10
2PM	4	6	8	14	7
3PM	5	3	6	9	4.5
4PM	6	0	3	3	1.5
			0	0	0

River flow at 1PM = DRH + baseflow

$= \text{Rainfall access} \times UH + \text{baseflow}$

$= 10 \times 4 + 20$

$= 60 \, m^3/s$

cm এ যাবেনা

flow value m^3/s এই আসবে

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* ~~If baseflow of 2hr Unit hydrograph is~~

* If base period of 2hrs unit hydrograph is 10 hours, calculate the base period of 8hr unit hydrograph.

Soln: base period of 8hrs unit hydrograph

$= T_{B_1} + (n-1) * UH_1$

$= 10 + \left(\frac{8}{2} - 1\right) * 2$

$= 16$ hours

$10 + (8-2) = 16$ hrs

* T_B of 3hrs UH is 12 hours. T_B of 9hrs UH is ______?

Soln: $T_{B_2} = T_{B_1} + (n-1) * UH_1$

$= 12 + \left(\frac{9}{3} - 1\right) * 3$

$= 12 + 2 \times 3 = 18$ hrs

$12 + (9-3) = 18$ hrs

* T_B of 4hr UH → 16 hours

T_B of 8hrs UH $= 16 + \left(\frac{8}{4} - 1\right) \times 4$

$= 20$ hrs

$16 + (8-4) = 20$ hrs

Maximum Flood Estimation

Rational method:

$$Q = \frac{CIA}{3.6}$$

Q = discharge (m^3/sec)

C = runoff coefficient

I = intensity (mm/hr)

A = Area (km^2)

$$Q = \frac{CIA}{360}$$ if A in hectare ; I in mm/hr

runoff volume, yield = CAP

P = Precipitation (rainfall depth)

yield = runoff volume (m^3)

Time of concentration: The time of concentration of a drainage basin is the time required by the water to reach the outlet from the most remote point of the drainage area.

Problems BWDB 2016

(1) The value of total direct runoff volume of catchment $6 \times 10^6 m^3$. If rainfall depth is 8 cm & runoff coefficient 0.5. Determine the catchment area in km^2.

Soln:

Yield = CAP

$6 \times 10^6 = 0.5 \times A \times \frac{8}{100}$

$\Rightarrow A = 15 \times 10^7 m^2$

$\Rightarrow A = 150 \ km^2$

(2) Calculate ϕ-index of rain water available of roof area $30 m^3$ for a family having a roof of $20 m^2$. Assume $C = 0.70$.

Soln:

Yield = CAP

$\Rightarrow P = \frac{\text{Yield}}{CA}$

$= \frac{30}{0.7 \times 20}$

$= 2.14 m$

ϕ index $= 0.3 \times 2.14$

$= 0.642 m/year$

* assume 1 year time duration

* 0.70 of Rainfall runoff

225 বাকী - 0.3 infiltration

(3) Determine the rainwater availability for a roof area of 5cm×5cm. Intensity 25cm/year & runoff coefficient 0.70. (WASA'14)

Sol^n:

$A = 5 \times 5 = 25\ cm^2$

$I = 25\ cm/year$

Rainwater availability,

$Q = CIA$

$= 0.7 \times 25 \times 25$

$= 437.5\ cm^3/year$

(4) A catchment of area 200 hectare has a runoff coefficient 0.5. A storm duration larger than the time of concentration of the catchment and of intensity 3.6cm/h causes a peak discarge of?

Sol^n: $Q = \frac{CIA}{360}$

$= \frac{0.5 \times 36 \times 200}{360}$ (mm)

$= 10\ m^3/sec$

(5) Rainfall intensity 2.4 m/yr. Runoff coefficient 0.7. Find the minimum catchment area of the daily water demand is 15 lpcd of 7 people.

Soln:

$Q = 15\ lpcd^{-daily} \times 7$

$= 15\ litre/day \times 7$

$= 15 \times 10^{-3}\ m^3/day \times 7$

$= 105 \times 10^{-3}\ m^3/day$

$Q = CIA$

$\Rightarrow A = \frac{Q}{CI}$

$= \frac{105 \times 10^{-3} \times 365}{0.7 \times 2.4}$

$= 22.81\ m^2$ Ans

DNCC'16

(6) Rainfall were applied at constant rate to an ~~impervious~~ impervious surface area of 6 acre. Using rational method calculate peak rate of rainfall (in cfs). Intensity of rainfall 1.5 inch/hr

Solⁿ: ∵ impervious, $C = \frac{R}{P} = 1$

$Q = CIA$ | 1 acre = $43560 ft^2$

$\Rightarrow Q = 1 \times \frac{1.5}{12} \times 6 \times 43560$

$= \cancel{19620 ft^3/hr}$

$= 32670 ft^3/hr$

$= 9.075 ft^3/sec$

* A watershed of 1500 ha is discharging through a drain at an average rate of 2.5 m³/sec. Calculate the drainage coefficient. If the drainage coefficient is 3 cm/day what would be the discharge through the drain?

Solⁿ: (i) Drainage coefficient, $C = \frac{2.5 \times 3600 \times 24 \ m^3/day}{1500 \times 10^4 \ m^2}$

$= 0.0144 \ m/day$

$= 1.44 \ cm/day$ Ans

(ii) ~~Drainage coefficient~~, $\cancel{\frac{3}{100} = \frac{Q}{1500 \times 10^4}}$

$\therefore \cancel{Q = \frac{3}{100} \times 1500 \times 10^4}$

(ii) $Q = AC$

$= (1500 \times 10^4) \times \frac{0.03 \ m}{24 \times 3600}$

$= 5.21 \ m^3/sec$ Ans

$Q = A \frac{1}{n} A R^{2/3} S^{1/2}$

(4) A culvert is designed for a flood frequency of 100 years & a useful life of 20 years. Risk?

Solⁿ: $T = 100$, $n = 20$

$$\text{Risk} = 1 - q^n$$
$$= 1 - (1-P)^n$$
$$= 1 - \left(1 - \frac{1}{100}\right)^{20}$$
$$= 18.2\ \%$$

(5) The probability that the annual max^m flood discharge will exceed 25000 m^3/s, at least once in next 5 years is 0.25. What is the return period?

Solⁿ:

$$\text{Risk} = 0.25$$
$$\Rightarrow 1 - q^n = 0.25$$
$$\Rightarrow 1 - (1-P)^n = 0.25$$
$$\Rightarrow 1 - \left(1 - \frac{1}{T}\right)^n = 0.25$$
$$\Rightarrow 1 - \left(1 - \frac{1}{T}\right)^5 = 0.25$$
$$\Rightarrow T = 18 \text{ years} \quad \text{Ans}$$

(6) A bridge has an expected life of 25 years and is designed for a flood magnitude of return period 100 years. What is the risk of the hydraulic design? If a 10% risk is acceptable what return period will have to be adopted? WRE 15

Sol^n: $n = 25$ years

$T = 100$

$P = 0.01$

$$\text{Risk} = 1 - q^n$$

$$= 1 - (1-P)^n$$

$$= 1 - (1-0.01)^{25}$$

$$= 0.22$$

$$= 22\%$$

Part b : Risk $= 10\%$

$\Rightarrow 0.10 = 1-(1-P)^{25}$

$\Rightarrow (1-P)^{25} = 1-0.10 = 0.9$

$\Rightarrow 1-P = 0.996$

$\Rightarrow P = 0.004$

$\Rightarrow T = 237.78$ years

(8) Determine the percentage chance that a 25 years storm may occur

(i) In the next 10 years (ii) In the next year itself

(iii) May not occur in another 15 years

Solⁿ:

(i) Risk $= 1 - q^n$

$= 1 - (1-P)^n$

$= 1 - \left(1 - \frac{1}{25}\right)^{10}$

$= 33.5\%$

(ii) Risk $= 1 - \left(1 - \frac{1}{25}\right)^{1}$

$= 4\%$

(iii) Reliability $= (1-P)^n = \left(1 - \frac{1}{25}\right)^{15}$

$= 54.21\%$

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* A culvert is designed for a flood frequency of 100 years and a useful life of 20 years. Calculate the risk involved in the design of the culvert.

Sol^n:

$$\text{Risk} = 1 - \left(1 - \frac{1}{T}\right)^n$$

$$= 1 - \left(1 - \frac{1}{100}\right)^{20}$$

$$= 0.182$$

$$= 18.2\%$$

* The probability that the annual max^m flood discharge will exceed 25000 m³/s, at least once in next 5 years is found to be 0.25. calculate the return period of this flood event.

Sol^n:

$$\text{Risk} = 1 - \left(1 - \frac{1}{T}\right)^n$$

$$\Rightarrow 0.25 = 1 - \left(1 - \frac{1}{T}\right)^5$$

$$\Rightarrow \left(1 - \frac{1}{T}\right)^5 = 1 - 0.25 = 0.75$$

$$\Rightarrow 1 - \frac{1}{T} = \sqrt[5]{0.75} = 0.94408$$

$$\Rightarrow \frac{1}{T} = 1 - 0.94408 = 0.056$$

$$\therefore T = 17.885 \text{ years}$$

(9) For a river valley project, the following results were obtained from flood frequency analysis using Gumbel's method.

Return Period	Peak flood (m^3/sec)
40	27000
80	31000

Estimate the flood magnitude with a return period of 240 years.

BWDB 2020, SGFL 23

Sol^n: Try solving using $K = 0.78 \ln T - 0.45$

$x_T = \bar{x} + (0.78 \ln T - 0.45)\,\sigma_{n-1}$

$x_{40} = \bar{x} + (0.78 \ln 40 - 0.45)\,\sigma_{n-1}$ ——(i)

$\Rightarrow 27000 = \bar{x} + (0.78 \ln 40 - 0.45)\,\sigma_{n-1}$ ——(ii)

$\Rightarrow 31000 = \bar{x} + (0.78 \ln 80 - 0.45)\,\sigma_{n-1}$

(ii) − (i) $\Rightarrow 4000 = \sigma_{n-1}\left(0.78 \ln \frac{80}{40}\right)$

$\Rightarrow \sigma_{n-1} = \frac{4000}{0.541} = 7398.436$

Now from eq^n (i), $\bar{x} = 27000 - (0.78 \ln 40 - 0.45) * 7398.436$

$= 9041.584$

$\therefore x_{240} = \bar{x} + (0.78 \ln 240 - 0.45) * 7398.436 = 37340 \ m^3/sec$

(12) The annual floods for a large period were statically analysed by Gumbel's method, which yielded $\overline{Q} = 19000$ cumec,

$\sigma = 3200$ cumec

Determine

(i) the probability of flood magnitude of 30,000 cumec occuring in the next year.

(ii) The flood magnitude of 5 yrs return period

Soln: (1)

$$30{,}000 = 19000 + (0.78 \ln T - 0.45) * 3200$$

$$\Rightarrow T = 146.05$$

$$\text{Risk} = 1 - q^n$$

$$= 1 - (1-P)^1$$

$$= 1 - \left(1 - \frac{1}{146.05}\right)$$

$$= 1 - 0.99$$

$$= 0.01 = 1\%$$

(ii) $$x_5 = 19000 + (0.78 \ln 5 - 0.45) * 3200$$

$$= 21572 \ m^3/sec$$ Ans

(13) Statistical analysis of the annual floods of the river Tapti using Gumbel's method yielded the 100-yr & 10 yr floods as 42800 & 22700 cumec respectively. Determine

(a) the magnitude of a 20 yr flood

(b) the probability of a flood magnitude 3500 cumec (i) occuring in next 10 years

(ii) in the next year itself

Sol^n:

$$42800 = \bar{x} + (0.78 \ln 100 - 0.45)\, \sigma_{n-1} \quad \text{— (i)}$$

$$22700 = \bar{x} + (0.78 \ln 10 - 0.45)\, \sigma_{n-1} \quad \text{— (ii)}$$

$$\text{(i)} - \text{(ii)} \Rightarrow 20{,}100 = \sigma_{n-1}\left(0.78 \ln \frac{100}{10}\right)$$

$$\Rightarrow \sigma_{n-1} = 11191.43$$

from eq^n (i),

$$\bar{x} = 42800 - (0.78 \ln 100 - 0.45) * 11191.43$$

$$= 7636.15$$

(a) $x_{20} = 7636.15 + (0.78 \ln 20 - 0.45) * 11191.43$ ~~= 28.85~~

$= 28750$ cumec

$= 28.750$ tcm

(b) $35000 = 7636.15 + (0.78 \ln T - 0.45) * 11191.43$

$\Rightarrow T = 41$ years

(i) occuring in next 10 years

$$\text{Risk} = 1 - (1-P)^{10}$$

$$= 1 - \left(1 - \frac{1}{41}\right)^{10}$$

$$= 22\%$$

(ii) occuring in next year

$$\text{Risk} = 1 - \left(1 - \frac{1}{41}\right)^{1}$$

$$= 2.44\%$$ Ans

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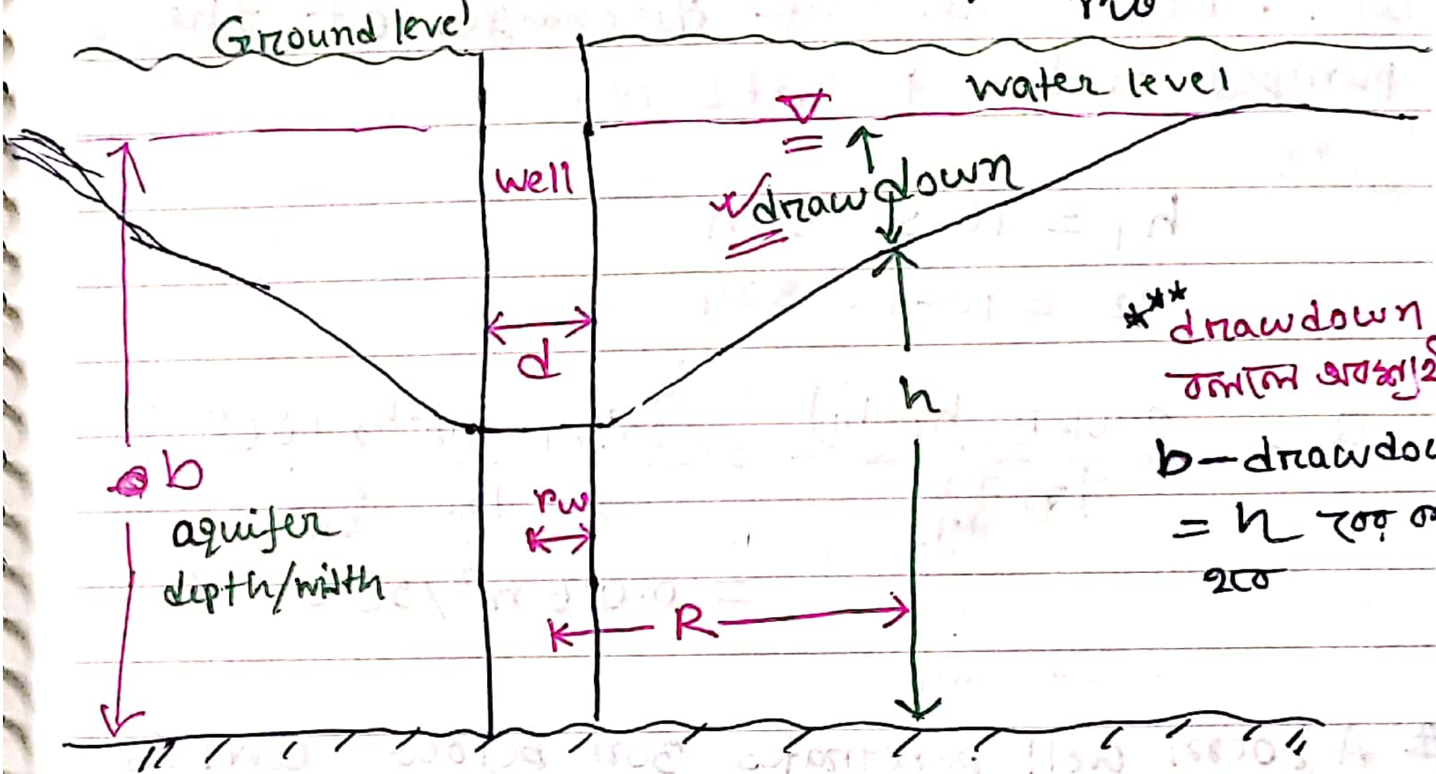
discharge in unconfined aquifer,

$$Q = \frac{\pi K (H^2 - H_w^2)}{\ln \frac{R}{r_w}}$$

discharge in confined aquifer,

$$Q = \frac{2\pi K b (H - H_w)}{\ln \frac{R}{r_w}}$$



** drawdown
তলানো অবস্থাই =

b − drawdown
= h বের করতে হবে

* T = Transmissibility (unit : mm^2/hr)
= Kb ← depth ~~width~~ of aquifer

K = hydraulic conductivity
unit mm/hr

* $\boxed{\text{artesian aquifer} = \text{confined aquifer}}$

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An artesian aquifer 10m thick with pizometric surface 40m above the bottom confining layer is being pumped by a fully penetrating well. The aquifer is medium sand ($k = 1.5\times10^{-4}$ m/s). Steady state drawdown of 5m & 1m are observed at two non-pumping wells located 20m & 200m respectively from the pumped well. Determine the discharge at the pumped well [SGFL' 17]

Soln:

$$h_1 = 10-5 = 5m$$
$$h_2 = 10-1 = 9m$$

$$Q = \frac{2\pi kB(h_2-h_1)}{\ln\frac{r_2}{r_1}} = \frac{2\pi\times1.5\times10^{-4}\times10(9-5)}{\ln\frac{200}{20}}$$
$$= 0.016\ m^3/sec$$ Ans

A 20cm well penetrates 30m below GWT. After a long period of pumping at a rate of 1800 lpm the drawdowns in the observation wells at 12m & 36m from the pumped well are 1.2m & 0.5m respectively. Determine transmissibility of the aquifer.

Soln:

$$Q = \frac{\pi k(h_2^2-h_1^2)}{\ln\frac{r_2}{r_1}}$$

$h_1 = (30-1.2)m = 28.8m$

$h_2 = (30-0.5)m = 29.5m$

$$\Rightarrow \frac{1.8}{60} = \frac{\pi k(29.5^2-28.8^2)}{\ln\frac{36}{12}}$$

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$\Rightarrow k = 2.57 \times 10^{-4}$ m/s
$= 22.2$ m/day

$\therefore T = KH$
$= 22.2 \times 30$
$= 666.14$ m²/day

এই math এ artesian কথাটাও নেই thickness ও বলা নেই, তারমানে এটা unconfined aquifer

---

A well fully penetrates a 25m thick confined aquifer. After a long period of pumping at a constant rate of 0.05m³/sec, the drawdown at a distance of 50m & 150m from the well were observed to be 3 & 1.2m. respectively. Determine hydraulic conductivity & transmissibility.

Soln:

$$0.05 = \frac{2\pi k \times 25\,(23.8 - 22)}{\ln\left(\frac{150}{50}\right)}$$

$\therefore k = 1.94 \times 10^{-4}$ m/sec

$T = 1.94 \times 10^{-4} \times 25 = 4.85 \times 10^{-3}$ m²/sec

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A 100mm diameter tube well is sunk 40m below static groundwater level. The depth & water in tubewell while pumping is 33m. The radius of drawdown is 30m & the coefficient of permeability of the aquifer = 0.50 lps/m^2. Calculate the [BWDB'22]

Soln :

$$Q = \frac{\pi k (h_w^2 - h^2)}{\ln \frac{R}{r_w}}$$

$$= \frac{\pi \times 0.50 \times 10^{-3} \times (40^2 - 33^2)}{\ln \left(\frac{30}{0.05}\right)}$$

$$= 0.125\ m^3/s$$

$r_w = \frac{100}{2}$

$= 50mm$

$= 0.05m$

$k = \frac{0.50 \times 10^{-3}\ m^3/s}{m^2}$

$= 0.50 \times 10^{-3}\ m/s$

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A tube-well of 20cm dia fully penetrates a horizontal, homogeneous and isotropic confined aquifer of infinite horizontal extent. The aquifer is of 30m uniform thickness. A steady pumping at the rate of 40litres/sec from the well for a long time results in a steady drawdown of 4m at the well face. The subsurface flow to the well due to pumping is steady, ~~horizontal~~ and the radius of influence line is 245m. Calculate the hydraulic conductivity of the ~~aq~~ aquifer in m/day.

Soln:

$$Q = \frac{2\pi k b (h_2 - h_1)}{\ln\left(\frac{R}{r}\right)}$$

drawdown

$$\Rightarrow 40 \times 10^{-3} m^3/s = \frac{2\pi \times k \times 30 \times 4}{\ln\left(\frac{245}{\frac{0.2}{2}}\right)}$$

dia = 0.2m
$\therefore r = \frac{0.2}{2}$

$$\Rightarrow k = 4.14 \times 10^{-4} m/s$$

$$\Rightarrow k = 4.14 \times 10^{-4} \times 60 \times 60 \times 24 \ m/day$$

$$\Rightarrow k = 35.77 \ m/day$$ A

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* A 100mm diameter tube well is sunk to 35m below in an unconfined aquifer. The depth of water in the tube well while pumping is 33m and the radius of influence is 30m. If the discharge is 33.4 lps, calculate the co-efficient of permeability of the aquifer. [WARPO'2017]

Soln:

$$Q = \frac{\pi k (H^2 - H_w^2)}{\ln\left(\frac{R}{r_w}\right)}$$

$Q = 33.4\ l/s$

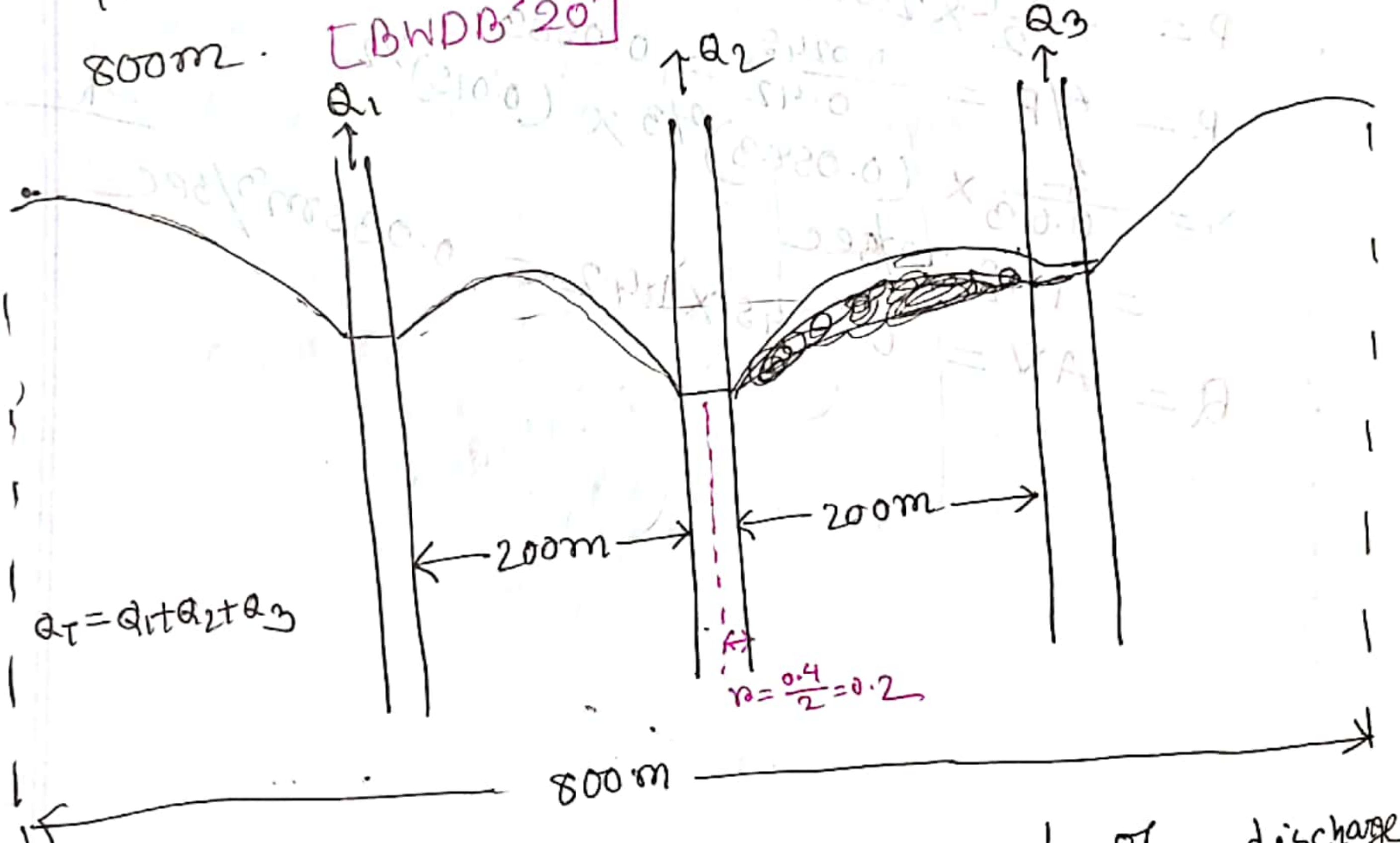
$= 33.4 \times 10^{-3}\ m/s$

$$\Rightarrow 33.4 \times 10^{-3} = \frac{\pi k \times (35^2 - 33^2)}{\ln\left(\frac{30}{\frac{0.1}{2}}\right)}$$

$$\Rightarrow k = 5 \times 10^{-4}\ m/s = 3\ cm/min$$

CamScanner

* Three pumping wells located along a straight lines are spaced at 200m apart. What should be the steady-state pumping rate from ~~which~~ each well so that the near steady-state drawdown in each well will not ~~exceed~~ exceed 2m? The transmissibility of the confined aquifer is 2400 m^2/day and all the wells are 40cm in dia. The thickness of the aquifer is 40m and the radius of influence of each well is 800m. [BWDB'20]



~~The~~ For the same amount of dischage (Q_T), draw down in the middle well (2) will

$\boxed{T = KB}$

be more. Therefore, the drawdown in this well governs.

The drawdowns in well 2 due to pumping in wells 1, 2 & 3 are respectively.

$$\boxed{Q = \frac{2\pi T S_w}{\ln \frac{R}{r_0}}}$$

$$S_{22} = \frac{Q}{2\pi T} \ln \frac{800}{0.2}$$

$$= \frac{Q}{2\pi \times \frac{2400}{24}} \ln \frac{800}{0.2}$$

$$= 0.0132\, Q$$

$$S_{21} = \frac{Q}{2\pi \times \frac{2400}{24}} \ln \frac{800}{200} = 0.0022\, Q$$

$S_{21} = S_{23} = 0.0022Q$

$\therefore$ Total drawdown in well (2), $S = 0.0132 + 0.0022Q + 0.0022Q$

$= 0.0176Q$

$\therefore Q = \dfrac{2}{0.0176} = 113.55\ m^3/hr$ Ans

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* Residence time is the average duration of a particle of water to pass through a phase of the hydrological cycle.

[BIWTA '23]

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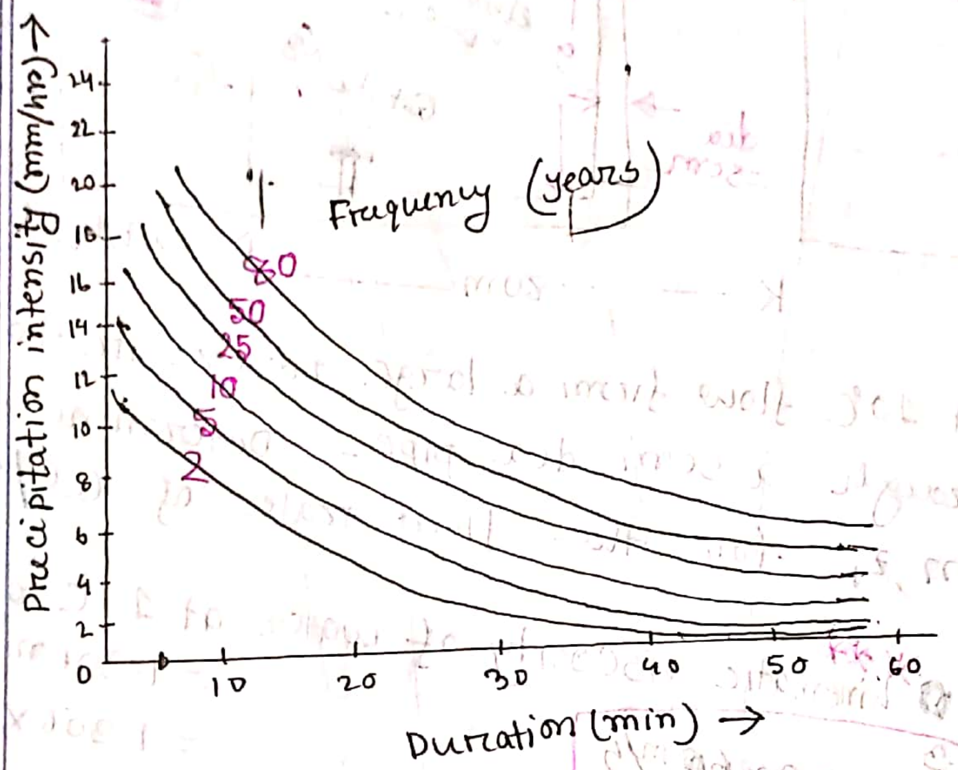
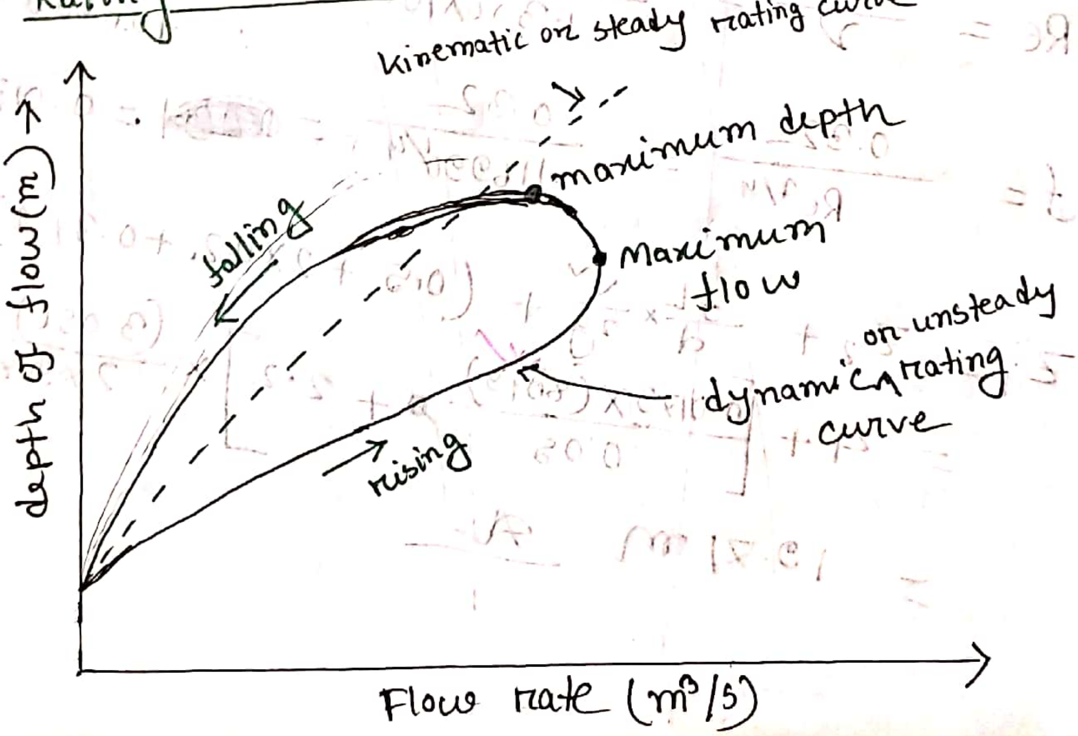
Intensity - Duration - Frequency (IDF) curves:

[DWASA'21]



Rating Curve of Unsteady flow: [DWASA'21]



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* Why using a manometer is important?

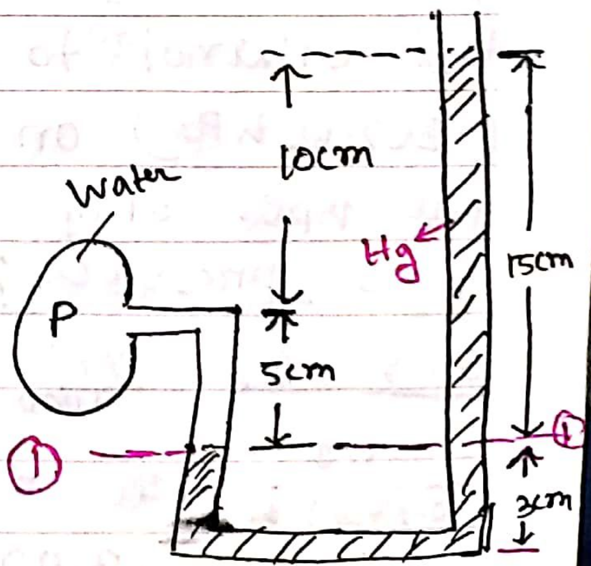
A manometer is an instrument which is widely used for many industrial applications, this device can be used to measure the pressure difference between two points, in a pipe or it can also be used to determine the pressure difference between two pipes.

* A U-tube manometer containing Hg (sp.gr 13.6) has its right limb open to the atmosphere, the left limb is full of water and is connected to a pipe containing water under pressure. Find the pressure of water in the pipe above the atmosphere for the manometer below —

Sol^n: At section (1-1)

$$P + \frac{5}{100} \times 9.81 = \frac{15}{100} \times 13.6 \times 9.81$$

$$\Rightarrow P = 19.52 \ KN/m^2$$



* A rectangular tank of 5m length, 3m width and 2.5m height. Calculate pressure force at the bottom of the tank. [BPDB'21]

Soln: Force, $F = A\gamma h$

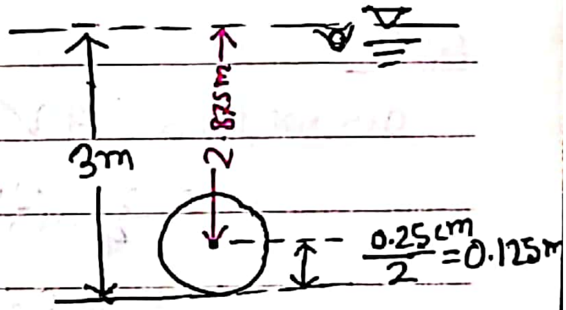
$$= (5\times3)\times 9.81\times 2.5 = 367.875 \text{ N}$$

\# What is the resultant force on one side of a 25cm diameter vertical circular plate standing at the bottom of 3m pool of water? [SGFL'17]

Soln: $F = A\gamma\bar{x}$

$$= \frac{\pi}{4}\times(0.25)^2\times 9.81\times 2.875$$

$$= 1.38 \text{ KN}$$



\# Determine the total pressure on a circular plate of diameter 1.5m which is placed vertically in water in such a way that the centre of the plate is 3m below the free surface of water. Find the position of centre of pressure also.

Soln: $F = \frac{\pi}{4}\times(1.5)^2 * 9.81 * 3 = 52\text{KN}$

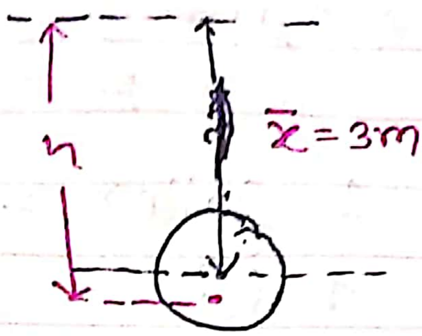
P.T.O

Position of centre of pressure,

$$h = \frac{I_G}{A\bar{x}} + \bar{x}$$

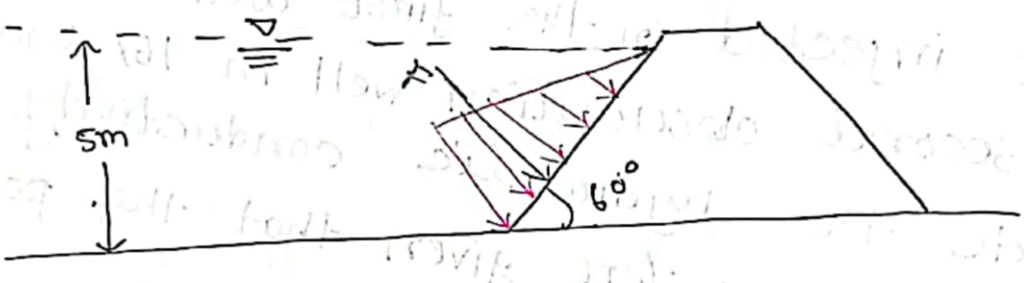
$$= \frac{\frac{\pi}{64}*(1.5)^4}{\frac{\pi}{4}\times(1.5)^2 * 3} + 3$$

$$= 3.05\text{m} \quad \text{Ans}$$



অর্থাৎ, এই force টি তার কেন্দ্রের থেকে একটু নিচে কাজ করে।

* Determine the hydrostatic force and the location of the center of pressure on the 25m long dam shown bleow. The face of the dam is at an angle of 60°.



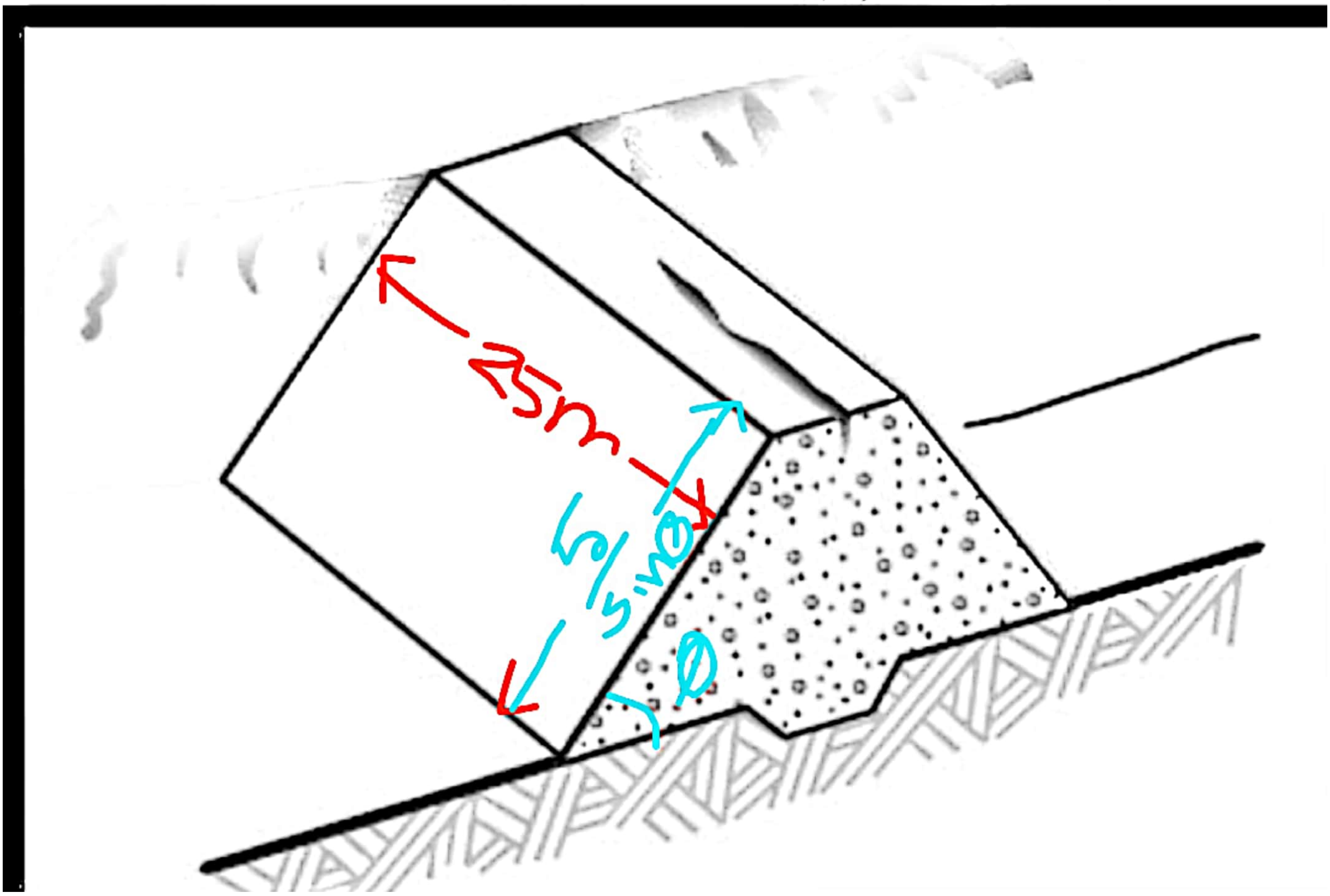
Solⁿ: $A = 25 \times \frac{5}{\sin 60°} = 144.3376 m^2$

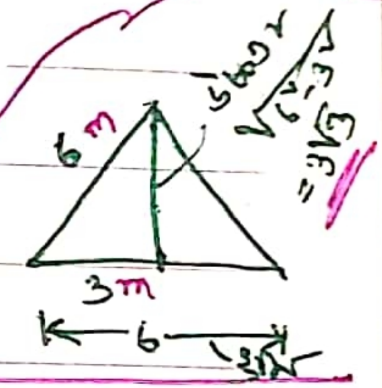
Force, $F_1 = \gamma \bar{x} A = 9.81 \times \frac{5}{2} \times 144.3376$

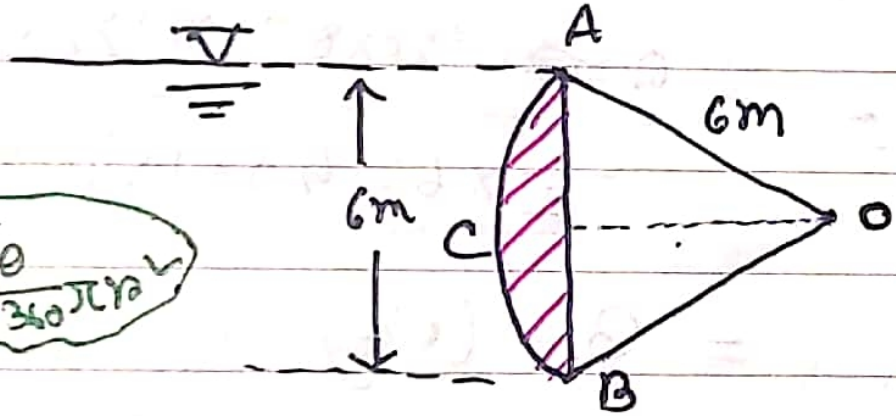
$= 3539.88 KN$

$= $ ~~3539.881~~

Center of pressure $= \frac{2}{3} \times 5 = 3.33m$



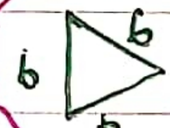


* 

Calculate resultant force

[BEPZA'16]

area of arc $= \frac{\theta}{360}\pi r^2$



with same Δ
$\theta = 60^\circ$

Solⁿ: $F_x = \frac{1}{2}\gamma h^2 = \frac{1}{2} \times 9.81 \times 6^2 = 176.58$ KN

F_y = ওজন
$= V\rho g$
$= V\gamma$

area × width

F_y = Self weight of water of ABC equivalent ~~area~~ volume

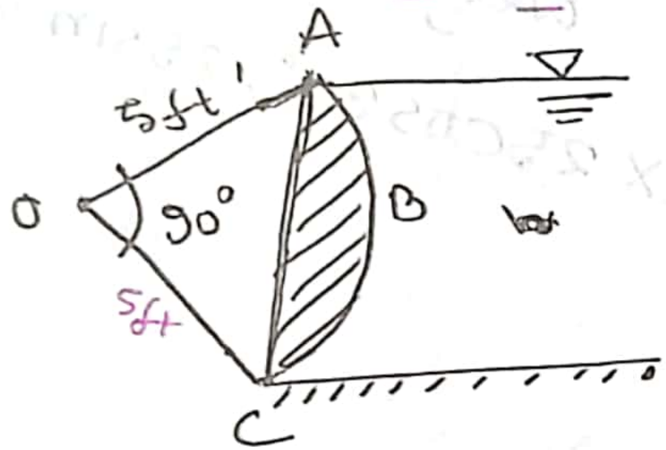
= width [area of arc OACB − area of OAB] * 9.81

Taking 1m width

$= 1 \times \left[\frac{60}{360} * \pi \times 6^2 - \frac{1}{2} \times 6 \times 3\sqrt{3} \right] * 9.81$

$= 32$ KN

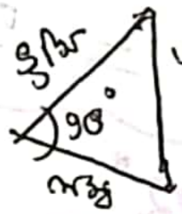
∴ Resultant, $R = \sqrt{(176.58)^2 + (32)^2} = 179.45$ KN, $\theta = \tan^{-1}\frac{32}{176.58} = 10.27^\circ$

* 

Curved wall ABC
width 9ft
compute horizontal, vertical & Resultant force.

Soln:

$$F_H = \gamma A \bar{x} = 62.43 \times (7.07 \times 9) \times \frac{7.07}{2}$$
$$= 14046.75 \text{ lb}$$



$= \sqrt{5^2+5^2}$
$= 7.07$

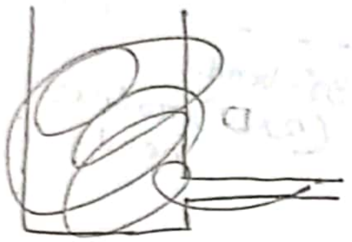
$\boxed{\text{Arc AOCB} = \frac{\text{circle}}{4}}$

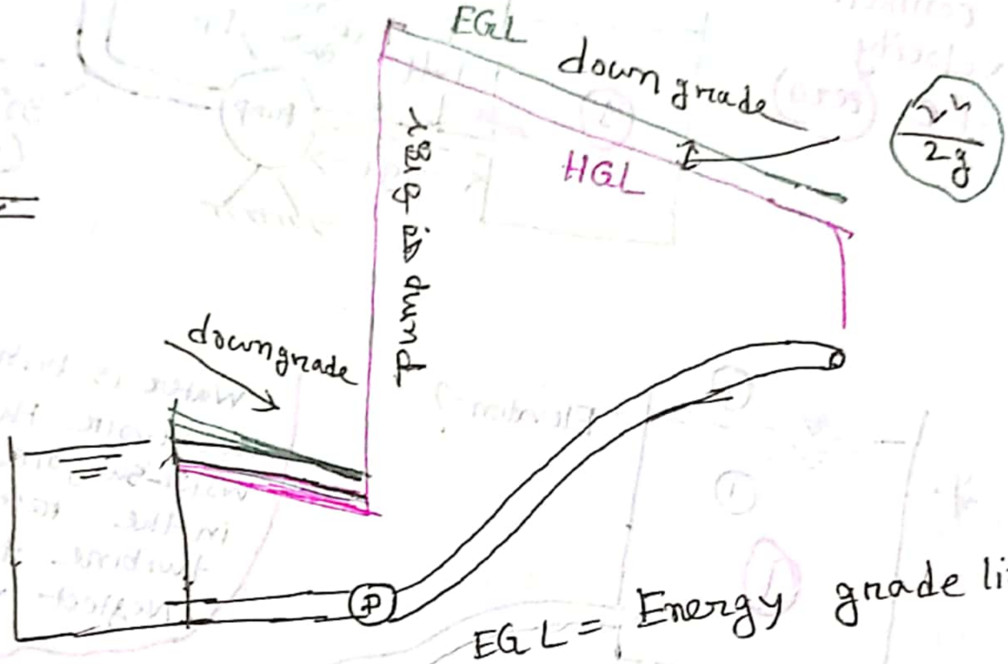
F_V = weight of (imaginary) water in hatched area

$= \text{unit weight} \times [\text{Arc AOCB} - \Delta AOC] \times \text{width}$

$= 62.43 \times \left[\frac{\pi \times 5^2}{4} - \frac{1}{2} \times 5 \times 5\right] \times 9 = 4008.91 \text{ lb}$

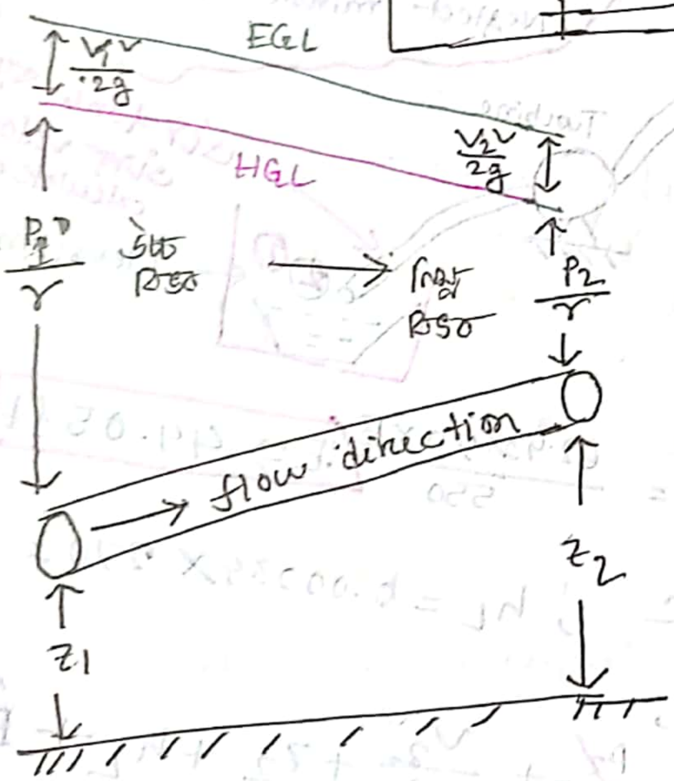
উচ্চচাপ → নিম্ন চাপ

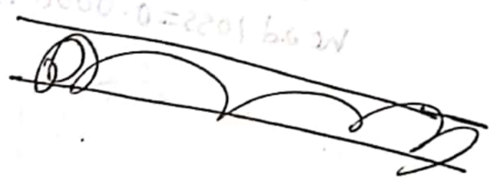




EGL = Energy grade line

HGL = ~~Hydrogen~~ Hydraulic grad line





Pizometer : is what? uses?

→ Pizometer is a device which measures the height at which a column of liquid rises against gravity in a system.
In geotechnical engineering, pizometer is used to measure pore water pressure in the soil, earth/rock fills, foundations & concrete structures.

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* The reading of pitot tube and piezometer are 40cm and 30cm respectively. calculate the velocity of the pipe. [BIWTA'23]

Soln:

Pitot tube = গতিজনিত head

Piezometer = Pore water pressure head

তাহলে, Velocity head = (Pitot tube − Piezometer reading)

$$\Rightarrow \frac{v^2}{2g} = (40-30)\,cm = 0.1m$$

$$\therefore v = \sqrt{2\times9.81\times0.1} = 1.4ms^{-1}$$

A water reservoir is under a pressure of 490 kN/m². Find the theoritical rate at which water in it is escaped through a circular orificice of 2.5cm dia.

Soln:

$$H = \frac{P}{\gamma} = \frac{490}{9.81} = 49.95m$$

Orifice এর ক্ষেত্রে

$$\therefore Q = A\sqrt{2gh}$$

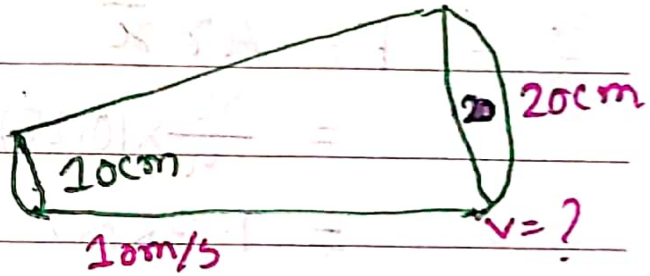
$$= \frac{\pi}{4}\times\left(\frac{2.5}{100}\right)^2\times\sqrt{2\times9.81\times49.95}$$

$$= 0.0154\,m^3/s$$ A

A circular pipe with 10 cm diameter having flow with an average velocity of 10 ms^{-1}. Calculate the discharge in lps. Also calculate the velocity of another end of the pipe with gradual diameter of 20 cm. [BPDB'21]

Soln:

discharge, $Q = AV$



$$= \frac{\pi}{4} \times \left(\frac{10}{100}\right)^2 \times 10$$

$$= 0.0785 \ m^3/s$$

$$= 0.0785 \times 10^3 \ lps$$

$$= 78.5 \ lps$$

Discharge throughout the pipe is constant.

$\therefore A_1 v_1 = A_2 v_2$

$$\Rightarrow v_2 = \frac{A_1}{A_2} \times v_1$$

$$= \frac{10^2}{20^2} \times 10 = 2.5 \ m/s \ \text{Ans}$$

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A pipe discharge $40 m^3/s$. Head loss is 10% of the velocity head difference. Find out pressure at point B.

[BWDB'18]

	Point A	Point B
elevation	1011m	1050m
pipe dia	50cm	100cm
pressure	50 KPa	?

Soln:

$$Z_A + \frac{P_A}{\gamma} + \frac{V_A^2}{2g} = Z_B + \frac{P_B}{\gamma} + \frac{V_B^2}{2g} + h_L$$

$$\Rightarrow \cancel{\frac{P_B}{\gamma} = (Z_A - Z_B) + \left(\frac{V_A^2 - V_B^2}{2g}\right) + \frac{P_A}{\gamma}}$$

$$\Rightarrow \frac{P_B}{\gamma} = (Z_A - Z_B) + \frac{V_A^2 - V_B^2}{2g} + \frac{P_A}{\gamma} - h_L$$

$$\Rightarrow \frac{P_B}{\gamma} = (Z_A - Z_B) + \left(\frac{V_A^2 - V_B^2}{2g}\right) + \frac{P_A}{\gamma} - 0.1\left(\frac{V_A^2 - V_B^2}{2g}\right)$$

$$\Rightarrow \frac{P_B}{\gamma} = (1011 - 1050) + 0.9 \times \frac{(203.72)^2 - (50.93)^2}{2 \times 9.81} + \frac{50}{9.81} = 1750.86$$

$\therefore P_B = 1750.86 \times 9.81 = 17176$ KN Ans

bernoulli's সমীকরণ এর মত করে formula লিখে নেবো, সুন্দর হবে

$$V_A = \frac{40}{\frac{\pi}{4} \times 0.5^2} = 203.72 m/s \qquad V_B = \frac{40}{\frac{\pi}{4} \times 1^2} = 50.93 m/s$$

অবশ্যই অবশ্যই অবশ্যই আগে $v = \frac{Q}{A}$ দিয়ে velocity বের করে নিতে হবে। নয়তো ভুল হবে!!

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A pipe lying at a head of 625 in one end P and other end at elevation of 520m. Diameter of P & Q are 50cm & 75cm respectively. Discharge from P to Q is 0.02 m³/sec. If the pressure at P is 1000 kN/m², determine the pressure at Q. Head loss is 1m [DWASA'17][BWDB'16] [MSc WRE'1?]

Soln:

$$V_P = \frac{0.02}{\frac{\pi}{4}\times 0.5^2} = 0.102 \text{ m/s}$$

$$V_Q = \frac{0.02}{\frac{\pi}{4}\times 0.75^2} = 0.045 \text{ m/s}$$

$$\therefore Z_P + \frac{V_P^2}{2g} + \frac{P_P}{\gamma} = Z_Q + \frac{V_Q^2}{2g} + \frac{P_Q}{\gamma} + h_L$$

$$\Rightarrow \frac{P_Q}{\gamma} = (625-520) + \frac{0.102^2 - 0.045^2}{2\times 9.81} - 1 + \frac{1000}{9.81}$$

$$\Rightarrow \frac{P_Q}{9.81} = 205.94$$

$$\Rightarrow P_Q = 2020.24 \text{ kN/m}^2 \text{ Ans}$$

* Area of point A is 20 cm^2 and area of point B is 80 cm^2. If height B from A is 0.5 m, headloss 0.05 m and pressure of point A = 50 Pa, ~~Q = 80 l/sec~~ discharge from point A to B is is 80 l/sec, find pressure at point B [CPGCBL'22] [PGCB'21]

Soln: $A \longrightarrow B$ flow

$$\frac{Q^2}{2gA^2} = \frac{v^2}{2g}$$

$\therefore$ headloss হবে B তে যাবে।

$$\underbrace{\frac{50}{9.81\times1000}}_{\text{kN to N}} + \frac{(80\times10^{-3})^2}{2\times9.81\times(20\times10^{-4})^2} = \frac{P_B}{9.81\times1000} + \cancel{\frac{(80\times10^{-3})^2}{2\times9.81\times(80\times10^{-4})}}$$

$$+ \frac{(80\times10^{-3})^2}{2\times9.81\times(80\times10^{-4})^2} + 0.5 + 0.05$$

$\Rightarrow P_B = 744654.5$ Pa

$= 744.65$ kPa

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hose pipe

(4) Water is flowing in a fire ~~nozzle~~ with a velocity of 1 m/s & pressure of 200 kPa. At the nozzle, the pressure decreases to atmospheric pressure (101.325 kPa), there is no change in height. calculate the velocity of the water exiting the nozzle. [ELDL 2022]

Solⁿ:

$$\frac{200}{9.81} + \frac{1^2}{2\times 9.81} = \frac{101.325}{9.81} + \frac{v^2}{2\times 9.81}$$

$$\Rightarrow v = 14.012 \, m/s \quad \underline{A}$$

A pipe 15cm in dia carries oil (sp. gr. = 0.75) at a rate of 70 lps. At a section 12cm above datum, the pressure is equivalent to 2cm of mercury vacuum. If the kinetic energy correction factor for the section is 1.1, the total head at that section in meters of oil is? [BIWTA'19]

Solnº Total head $= z + \frac{P}{\gamma} + \alpha \frac{v^2}{2g}$

vaccuam তাই এটি ঋণাত্মক

$$= 0.12 - 0.02\left(\frac{13.6}{0.75} - 1\right) + 1.1 \times \frac{\left[\frac{70\times10^{-3}}{\pi/4 \times 0.15^2}\right]^2}{2\times9.81}$$

$= 0.66\text{m}$ ~~of~~ of oil

~~SG of heavy liquid (test)~~

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$$\text{pressure head} = \text{gauge difference} \times \left[\frac{\text{test liquid SG(heavy)}}{\text{pipe liquid}} - 1\right]$$

$$= \text{gauge diff} \times \left[1 - \frac{\text{SG of test liquid (light)}}{\text{pipe liquid}}\right]$$

test liquid সবসময় ই উপরেই থাকবে, 1 থেকে এটিকে বিয়োগ হবে।

* Releted Math

Through a refinery, fuel ethanol is flowing in a pipe at a velocity of 1m/s and a pressure of 101.325Pa. The refinery needs the ethanol to be at a pressure of 2atm (202.65kPa) on a lower level. How far must the pipe drop in height in order to achieve this pressure? (density of ethanol is 789kg/m³)

Solⁿ: $G_e = \frac{789}{1000} = 0.789$

$$\frac{101.325}{0.789 \times 9.81} = \frac{202.65}{0.789 \times 9.81} + h$$

$$\Rightarrow h = -13.09\ m$$

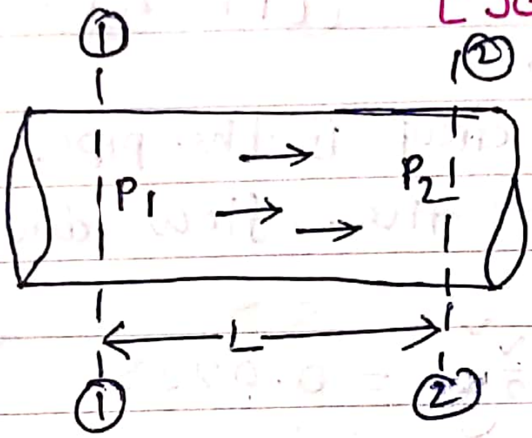
velocity দুই জায়গায় একই 1m/s, তাই extra ভাবে consider করার প্রয়োজন নাই।

– দিয়ে drop বোঝাচ্ছে।
13.09 m drop দিতে হবে।

Prove that, for constant discharge and constant ~~the~~ value of frictional resistance of a pipe, the head loss is inversely proportional to the 5th power of the pipe diamter

[JGTDSL'21] [SGFCL'21]

Soln:



Applying Bernoulli's eqn,

$$\frac{P_1}{\gamma} = \frac{P_2}{\gamma} + h_f$$

[∵ constant diameter]
[∵ constant velocity]

$$\Rightarrow h_f = \frac{P_1 - P_2}{\gamma}$$

$$\Rightarrow \Delta P = \gamma h_f$$

$$\Rightarrow \frac{F}{A} = \gamma h_f$$

Force = frictional resistance * wet area * velocity²

$$\Rightarrow \frac{f * \pi d L * v^2}{\frac{\pi}{4} \times d^2} = \gamma h_f$$

$$\Rightarrow h_f = \frac{4 f L v^2}{\gamma d} = \frac{4 f L \times \left(\frac{Q}{\pi/4 \times d^2}\right)^2}{\gamma d}$$

$$= \frac{64 f L Q^2}{\pi \gamma d^5}$$

$$\boxed{\therefore h_f \propto \frac{1}{d^5}}$$

Proved

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* ⇒ Oil with $\rho = 900 kg/m^3$ & kinematic coefficient of viscosity, $\nu = 0.00001 m^2/s$, flows at $q = 0.2 m^3/s$ through 500m of 200mm dia cast-iron pipe. Take, $f = 0.0225$

Determine — [ERL'22] [GTCL'22]

(i) The head loss

(ii) The pressure drop if the pipe slopes down at 1 in 10 in the flow direction

Solnⁿ:

(i) $h_f = f \frac{L}{d} \frac{v^2}{2g} = 0.0225 \times \frac{500}{0.2} \times \frac{\left[\frac{0.2}{\frac{\pi}{4} \times 0.2^2}\right]^2}{2 \times 9.81}$

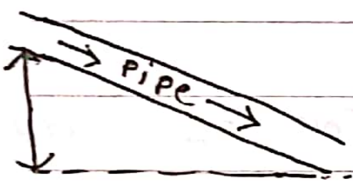
$= 116.2 m$

(b) $\Delta P = \Delta h \rho g$ [same dia pipe, velocity ~~discharge~~ change হবে না]

$= \left(116.2 - \frac{500}{10}\right) \times 900 \times 9.81$

$= 584479.8 N/m^2$

$= 584.48 KN/m^2$



$z_1 = 50$ $z_2 = 0$

এটার $z_1 + \frac{p_1}{\gamma} = z_2 + \frac{p_2}{\gamma} + h_f$ use হবে কোন ক্ষেত্রে,

তাই pressure, $p = h \rho g$ হবে

$h_f = f \frac{L}{d} \frac{v^2}{2g}$

মেকানিক্সের বই: ২টা মূল্য থাকলে f 0.02, 0.03, 0.04

মেকানিক্সের বই: 2টার মূল্য থাকলে $4f$ → 0.004, 0.008,

* A turbine is located at an elevation 200m below that of the surface of the water intake. The friction loss in the pipeline leading to it is 8m and the turbine efficiency is 90%. What is the power ~~input~~ delivered by the turbine if the flow is $3m^3/s$ in kW?

[MGMCL'22]

Soln: Power, $P = \eta \gamma Q h_f$

$= 0.9 \times 9.81 \times 3 \times (200+8)$ Plus 20

$= \cancel{5085.504 \text{ kW}}$ Ans

$= 5509.296 \text{ kW}$

* A reaction turbine is supplied with water through a 1500mm diameter pipe ($\lambda = 0.011$ mm) that is 50m long. The water surface in the reservoir is 27m above the draft-tube inlet that is 4.1m above the water level in the tailrace. If the turbine efficiency is 92% and the discharge is $12 m^3/sec$, what is the power output of the turbine in kW?

Soln: Head loss, $H_L = \lambda \frac{L}{D} \frac{v^2}{2g}$

পাম্প motor দিলে। তাই টার্বাইন এ H_L বিয়োগ হবে।

$= \frac{0.011}{1000} \times \frac{50}{1.5} \times \frac{\left\{\frac{12}{\pi/4 \times (1.5)^2}\right\}^2}{2 \times 9.81}$

$= 8.6177 \times 10^{-4}$

$\therefore P = \eta \gamma Q h_f$

$= 0.92 \times 9.81 \times 12 \times \left\{27 + 4.1 - (8.6177 \times 10^{-4})\right\} = 3368.11 \text{ kW}$

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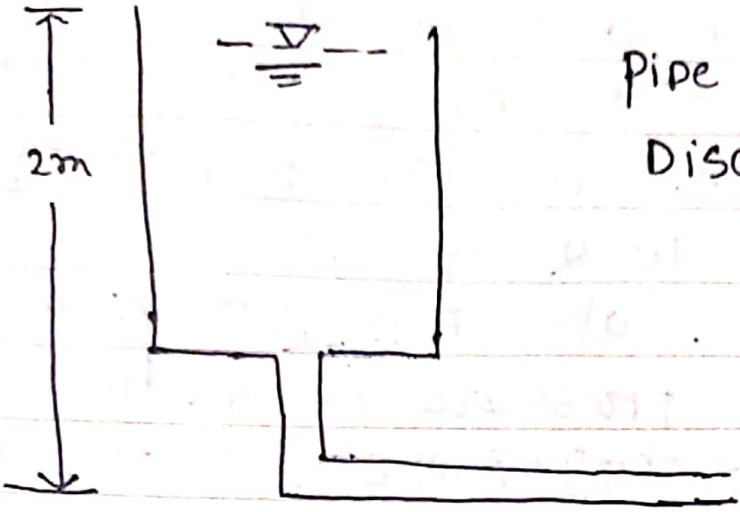


Pipe dia 1cm.

Discharge, Q = ?

[DSCC '19]

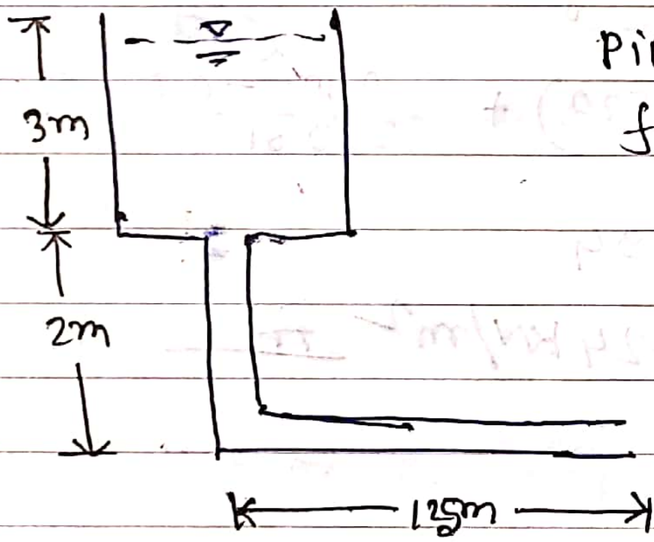
$v = \sqrt{2gh}$

Solⁿ: $Q = A\sqrt{2gh}$

$= \frac{\pi}{4} \times \left(\frac{1}{100}\right)^2 \times \sqrt{2 \times 9.81 \times 2}$

$= 4.92 \times 10^{-4}\ m^3/sec$

#



Pipe dia = 600mm

frictional factor, f = 0.04

calculate discharge

Solⁿ: $v = \sqrt{\frac{2gh}{1 + f\frac{L}{d}}} = \sqrt{\frac{2 \times 9.81 \times 5}{1 + 0.04 \times \frac{126.4}{0.6}}}$

$L = 125 + 2 - 0.6$
$= 126.4$

$= 3.226\ m/s$

$\therefore Q = \frac{\pi}{4} \times 0.6^2 \times 3.226 = 0.912\ m^3/s$

A

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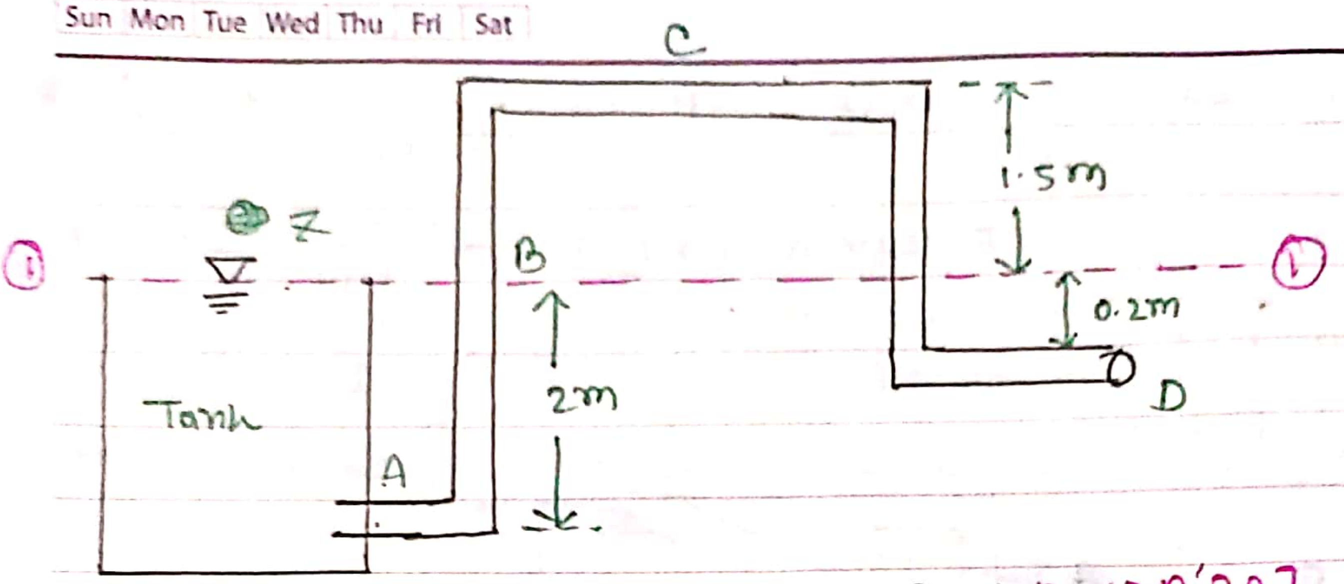


Find pressure at A, B, C, D [BWDB'20]

Solⁿ:

velocity at point D, $V = \sqrt{2 \times 9.81 \times 0.2} = 1.98\ m/s$

(পুরো pipe velocity = 1·98 কারণ uniform pipe এর velocity A থেকে যা D তেও তা,)

Applying Bernoulli's eqⁿ at point Z & C,

$$\Rightarrow 0 = 1.5 + \frac{P_C}{9.81} + \frac{1.98^2}{2 \times 9.81}$$

$$\therefore \boxed{P_C = -16.675\ KN/m^2}$$

Applying Bernoulli's eqⁿ at point Z & B,

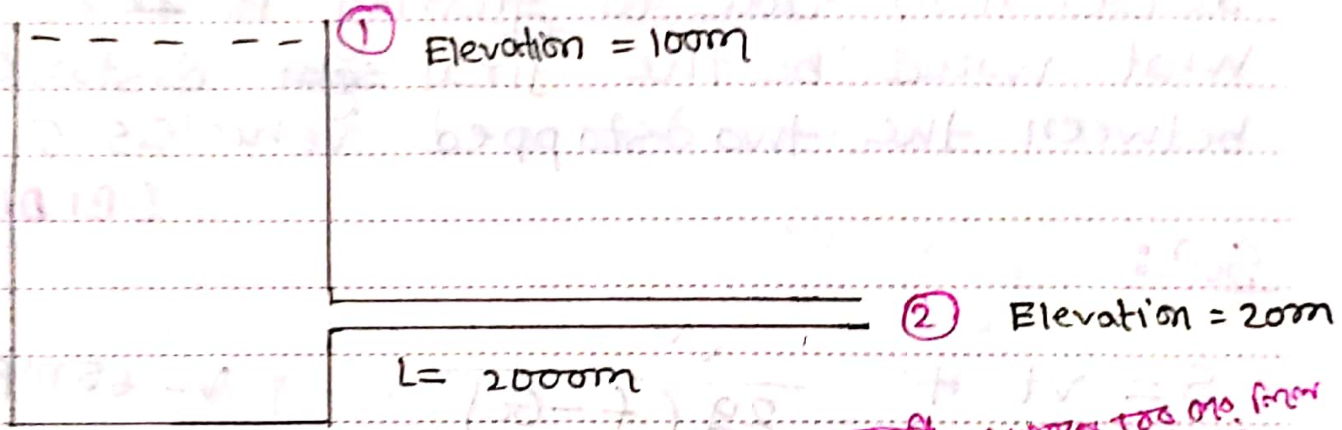
$$\Rightarrow 0 = \frac{P_B}{9.81} + \frac{1.98^2}{2 \times 9.81} \quad \therefore \boxed{P_B = -1.96\ KN/m^2}$$

At point A,

$$\Rightarrow 0 = -2 + \frac{P_A}{9.81} + \frac{1.98^2}{2 \times 9.81} \quad \therefore \boxed{P_A = 17.66\ KN/m^2}$$

Pressure at D, $\boxed{P_D = 0}$ [atmospheric pressure]

* The water flows from a thermal power plant through a pipe having diameter of 20cm. Flow ~~velocity~~ rate is $0.06 m^3/sec$ & frictional coefficient of the pipe is 0.02. Find the pressure at point 2. [BPDB'23]



V আগে বের করে নিলে ভুল হবে না।

Soln: velocity, $V = \dfrac{0.06}{\frac{\pi}{4}\times(0.2)^2} = 1.91 m/s$

*** V কে বের করে $\frac{Q}{A}$ আবার দিয়ে গুণ করো না

Now,

$$100 = \frac{P}{9.81} + \frac{1.91^2}{2\times9.81} + 20 + 0.02\times\frac{2000}{0.2}\times\frac{1.91^2}{2\times9.81}$$

$$\Rightarrow \frac{P}{9.81} = 42.6264$$

$\therefore P = 418.16$ KPa **A**

* Determine the coefficient of velocity of a small circular orificie in the side of a vessel. The orifice is 20cm belowe the vessel. The horizontal & vertical coordinates of water jet from venna contracta is 86cm & 96cm respectively.

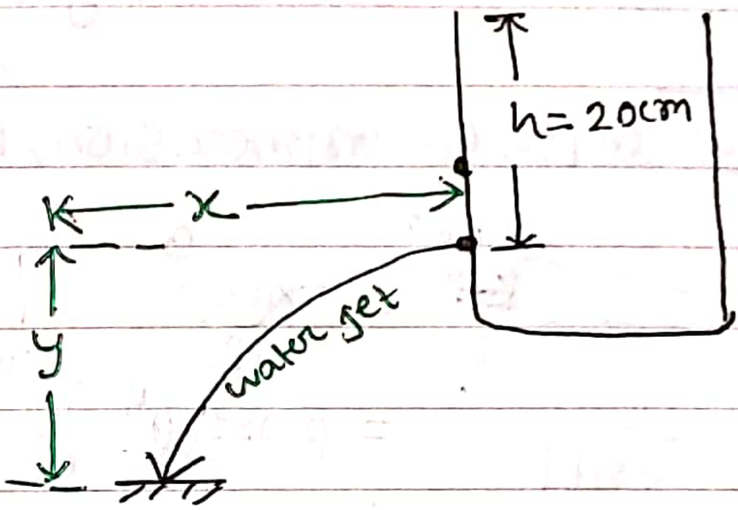
Solnº:

$x = C_v\sqrt{4yH}$

$\Rightarrow 86 = C_v\sqrt{4\times96\times20}$

$\therefore C_v = 0.98$

A



Similarly, coefficient of discharge $= C_d$

$Q = C_d \cdot A\sqrt{2gh}$; $C_d = \frac{C_c}{C_v}$

Calculate the total & effective pressure of swimming pool having 5m of water level from ground

[CPGCBL'18] [BPDB'16]

Solnº: Total pressure $= 9.81\times5 = 49.05m$

Effective pressure = Total pressure − water pressure

$= 0$ A

* A man is spraying water onto a car at a speed of 4m/s. The diameter of the water jet is 2.5cm. If the jet hits the car surface perpendicularly, calculate the force on the car surface due to the water jet. Neglect all loses. [RPGCL'22]

$= \rho Q V$

Soln: Thrust / Force $= \rho A V^2$

$= 1000 kg/m^3 \times \frac{\pi}{4} \times \left(\frac{2.5}{100}\right)^2 m^2 \times (4 m/s)^2$

$\frac{kg}{\cancel{m^3}} \times \cancel{m^2} \times \frac{\cancel{m^2}}{s^2}$ m

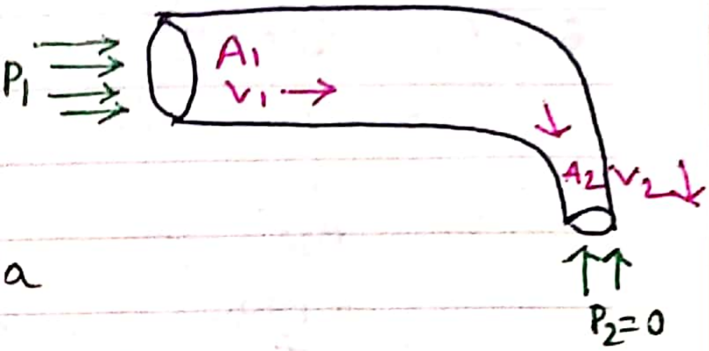
$\boxed{kg\,m\,s^{-2} = N}$

$= 7.854 N$

CamScanner

Water flows steadily through the 90° reducing elbow shown in the figure. At the inlet to the elbow, the absolute pressure is 220 kPa and the x-sectional area is 0.01 m². At the outlet the x-sectional area is 0.0025 m² and the velocity is 16 m/s. The elbow discharges to the atmosphere. Determine the force required to hold the elbow in place. [SGCL'17]

Soln:



$$V_1 = \frac{A_2}{A_1} \times V_2$$

$$= \frac{0.0025}{0.01} \times 16 = 4\,m/s$$

$$P_1 = P_{abs} - P_{atm}$$

$$= 220 - 101.325 = 118.675\,kPa$$

$$P_2 = 0$$

$$\overrightarrow{+}\ \sum F_x = \rho Q \Delta V$$

v_2 এর x উপাংশ zero

$$\Rightarrow R_x + P_1 A_1 = \rho Q (V_{2x} - V_{1x})$$

$$\Rightarrow R_x = -P_1 A_1 + \rho Q(-V_{1x})$$

যেহেতু force kN এ বের করবো তাই $\rho = 1$ হবে।

$$\Rightarrow R_x = -118.675 \times 0.01 - 1 \times (0.01 \times 4) \times 4$$

$$= -1.35\,KN \qquad \therefore R_x = 1.35\,KN\ (\leftarrow)$$

$$\downarrow^{+} \sum F_y = \rho Q (V_{2y} - V_{1y})$$

$$\Rightarrow R_y - P_2 A_2 = 1 \times (0.01 \times 4) \times 16$$

$$\Rightarrow R_y = 0.64\,KN\ (\downarrow)$$

[V_2 নিচের দিকে এবং নিচের দিক positive ধরেছি]

$$\therefore R = \sqrt{(-1.35)^2 + (0.64)^2} = 1.494\,KN \quad \text{Ans.}$$

ΔV = শেষ velocity − প্রথম velocity

যেমন $v_2 = 16$ শেষ তাই সবসময় v_2 আগে আসবে

Date :

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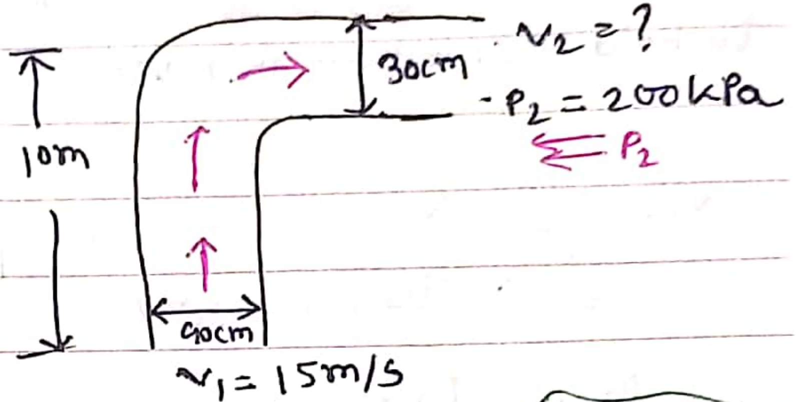
\# A 90° reducing elbow is in the vertical plane and water flows through it. what is the horizontal force required to hold the reducer elbow in a stationary position?



Soln:

$$Q = A_1 V_1 = \frac{\pi}{4} \times (0.4)^2 \times 15$$
$$= 1.885 \, m^3/s$$

$$\therefore V_2 = \frac{Q}{A_2}$$

$$= \frac{1.885}{\frac{\pi}{4} \times (0.3)^2} = 26.67 \, m/s$$

V_2 তো হবে
$V_2 - V_1$ হবে

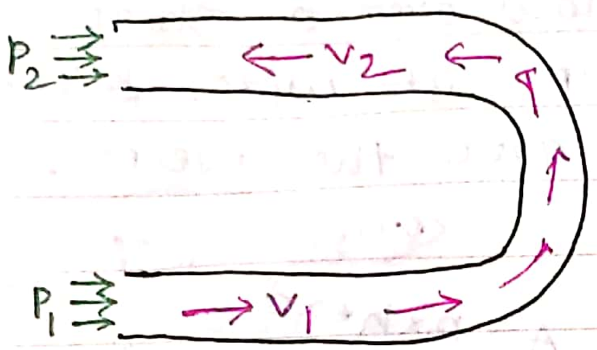
v_1 এর x উপাংশ zero

$$\sum \vec{F_x} = \rho Q (V_{2x} - V_{1x})$$

for kN unit $\rho = 1$

$$\Rightarrow R_x - P_1 A_1 = \rho Q V_{2x}$$

$$\Rightarrow R_x = 200 \times \frac{\pi}{4} \times (0.3)^2 + 1 \times 1.885 \times 26.67$$
$$= 64.41 \text{ KN } (\rightarrow) \quad \text{A}$$



$P_1 = 100\,kN/m^2$ Petrolium
$P_2 = 80\,kN/m^2$ (density $900\,kg/m^3$)
180° bend ~~[illegible]~~
flow rate $= 150\,m^3/hr$
pipe dia $= 200\,mm$

Determine force exerted by the liquid on the bend.

Solⁿ:

$$A = \frac{\pi}{4} \times 0.2^2 = 0.03\,m^2$$

$$Q = 150\,m^3/hr = \frac{150}{3600} = 0.04167\,m^3/s$$

$$v = \frac{0.04167}{0.03} = 1.389\,m/s$$

$\overset{+}{\rightarrow}$

$$\Sigma F_x = 0$$

$$\Rightarrow R_x + P_1A_1 + P_2A_2 + \rho Q(V_2 - V_1) = 0$$

kN → m²

$$\Rightarrow R_x + 100 \times 0.03 + 80 \times 0.03 + 0.9 \times 0.04167 \times [-1.389 - 1.389] = 0$$

direction ← v

$$\Rightarrow R_x + 5.3 = 0$$

$$\therefore R_x = -5.3\,kN$$

$$\therefore R_x = 5.3\,kN \; (\leftarrow)$$

* A nozzle is so shaped that the velocity of flow along the centre line changes linearly from A to B is 2.50m/sec to 16m/sec in a distance of 3.75m. Determine the magnitude of convective acceleration at point B.

Soln: [JOCL'22]

convenctive acceleration at point, $a_A = \frac{16-2.5}{3.75} \times 2.5$

$= 9 ms^{-2}$

at point B, $a_B = \frac{16-2.5}{3.75} \times 16$

$= 57.6 ms^{-2}$

ধরি যদি A point থেকে 2m দূরে convective acceleration বের করতে হবে।

∴ A প্রান্ত থেকে 2m দূরে velocity $= 2.5 + \frac{16-2.5}{3.75} \times 2$

$= 9.7 ms^{-1}$

∴ 2m দূরে acceleration $= \frac{16-2.5}{3.75} \times 9.7$

$= 34.92 ms^{-2}$

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If the velocity distribution over a plate is given by $u = \frac{2}{3}y - y^2$ in which u is the velocity in m/s at a distance y meter above the plate. Determine the shear stress at $y = 0.15$. Take dynamic viscosity of fluid as 8.63 poise.

[BWDB'16]

Solⁿ:

Here, $u = \frac{2}{3}y - y^2$ (1 poise = $0.1\ Ns/m^2$)

$\therefore \frac{du}{dy} = \frac{2}{3} - 2y$

at $y = 0.15$, $\frac{du}{dy} = \frac{2}{3} - 2\times 0.15 = 0.367$

Shear stress $= \mu \times \frac{du}{dy} = 0.1 \times 8.63 \times 0.367$

$= 0.3167\ N/m^2$

যদি pipe এর length 1000m এবং dia 4cm থাকে হলে, তাহলে pipe এ shear stress resistance

হবে $= 0.3167 \times \pi d l = 0.3167 \times \pi \times \left(\frac{4}{100}\right) \times 1000$

$= 39.8\ N$

A plate 0.025mm distant from a fixed plate moves at 60cm/s and requires a force of 2N per unit area to maintain this speed. Determine fluid viscosity between the plates.

Solⁿ: shear stress, $\tau = \frac{2N}{1m^2} = 2 N/m^2$

$dy = 0.025mm = 0.025 \times 10^{-3} m$

$du = 60 cm/sec = 0.6 m/sec$

We know, $\tau = \mu \frac{du}{dy}$ $\therefore \mu = \tau \times \frac{dy}{du}$

$\therefore \mu = 2 \times \frac{0.025 \times 10^{-3}}{0.6} = 8.33 \times 10^{-5} Ns/m^2$

$= 8.33 \times 10^{-4}$ poise

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\# A log of wood 6 in wide, 12 in high & 60 in long floats on the surface of water. Assuming that its specific gravity is 0.74. If the log floats on its 6-in side, determine —

(a) the depth of immersion

(b) The righting moment for a 5° tilt

(c) The metacentric height

Soln:

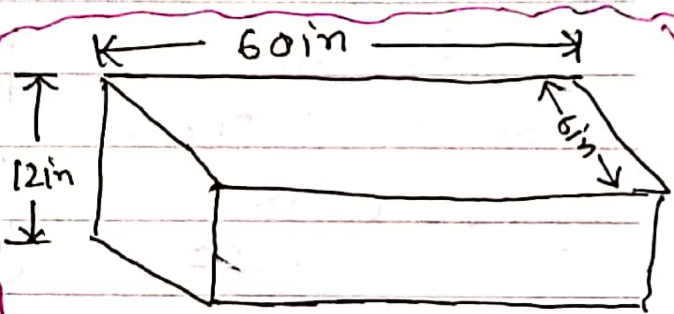
(a) depth of immersion, $h = 0.74 \times 12 = 8.88 \text{ in}$

(b) $I = \frac{l w^3}{12} = \frac{5 \times (0.5)^3}{12}$

$= 0.0521 \text{ ft}^4$

$\boxed{\begin{array}{l} 60 \text{ in} = 5 \text{ ft} \\ 6 \text{ in} = 0.5 \text{ ft} \end{array}}$

moment বের করতে depth দরকার নাকি ইউজ নিলে।



যেই side এ float করবে তার বিপরীত side'এ হবে depth

Righting moment for 5° tilt,

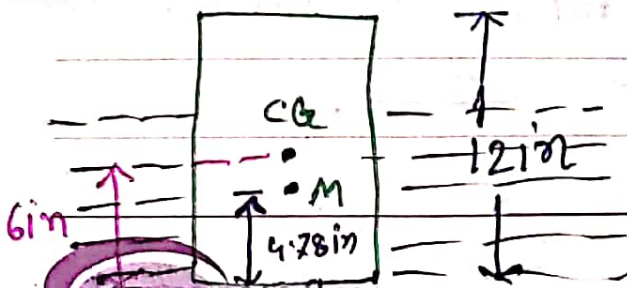
$M = 5^\circ * \gamma_w I$

$= \frac{5\pi}{180} \times 62.43 \times 0.0521 = 0.284 \text{ lb-ft}$

(c) Submerged volume, $V_{sub} = \frac{8.88 \times 6 \times 60}{12 \times 12 \times 12} = 1.85 \text{ ft}^3$

Metacentric height from bottom, $h_m = \frac{0.0521 \text{ ft}^4}{1.85 \text{ ft}^3} \times 12 + \frac{8.88}{2}$

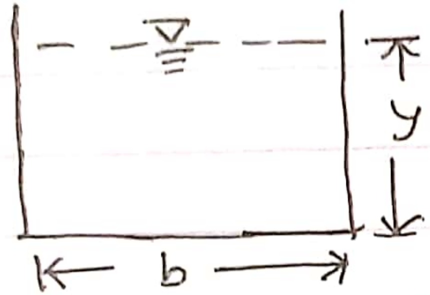
$= 4.78 \text{ in}$



Metacentric height CG এর উপরে থাকলে stable, নিচে থাকলে unstable.

∴ এটা unstable

Open Channel Shapes



Rectangular

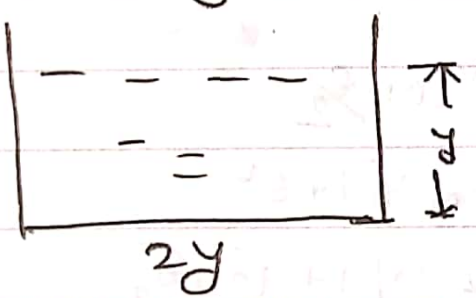
Area, $A = by$

Perimeter, $P = b + 2y$

Hydraulic Radius, $R = \frac{A}{P} = \frac{by}{b+2y}$

* For most efficient Rectangular channel.

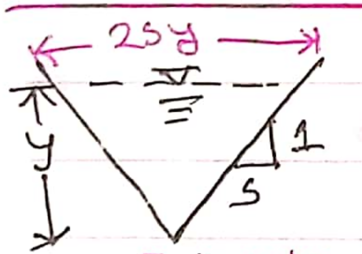
$b = 2y$



Area, $A = 2y^2$

$P = 4y$

$\therefore R = \frac{y}{2}$



Triangular

Top width $= 2Sy$ ***

$A = Sy^2$

$P = 2y\sqrt{1+S^2}$

$R = \frac{Sy^2}{2y\sqrt{1+S^2}}$

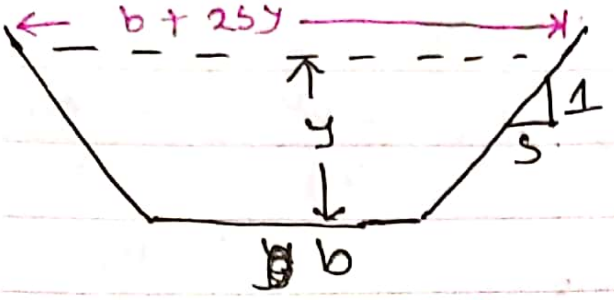
For most efficient ~~rectangular~~ Triangular channel, $S = 1$

$\therefore \boxed{A = y^2}$ ***

$R = \frac{y}{2\sqrt{2}}$

* S সবসময় Horizontal এর জন্য হবে। vertical সবসময় 1 হবে।

Triangle আর Trapezoid same. Just Trapezoid এ extra b আছে



Trapezoidal channel

$A = (b + sy)y$

$P = b + 2y\sqrt{1+s^2}$

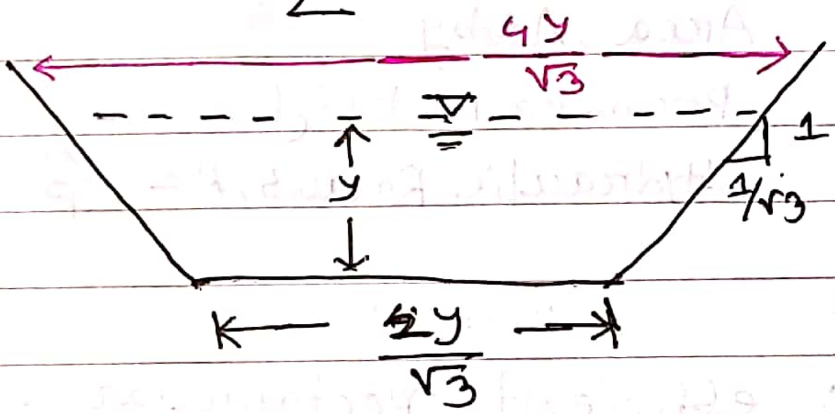
$R = A/P$

Top width $= b + 2sy$

For best trapezoidal channel, $s = \frac{1}{\sqrt{3}}$

$A = \sqrt{3}\,y^2$

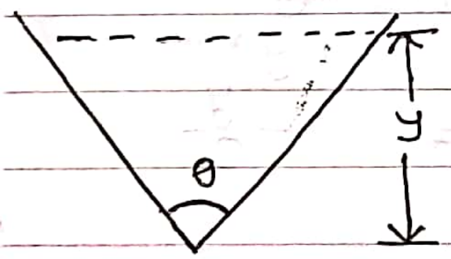
$R = \frac{y}{2}$



best section

angle দেওয়া থাকলে —

$* \; s = \tan\theta/2$



$A = y^2 \tan\theta/2$

$P = 2y\sqrt{1+s^2}$

$= 2y\sqrt{1+\tan^2\theta/2}$

$= 2y \sec\theta/2$

$= \frac{2y}{\cos\theta/2}$

Top width, $T = 2sy$

$= 2y\tan\frac{\theta}{2}$

$R = \frac{A}{P}$

* For best section,

$\theta = 90°$ ***

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Channel Design Formulae :

- Chezy's formula —

$V = C\sqrt{RS}$

* C = Chezy's coefficient

Manning's formula —

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad [\text{SI unit}]$$

$$= \frac{1.486}{n} R^{2/3} S^{1/2} \quad [\text{FPS unit}]$$

→ সংখ্যাগুলো ব্যবহারে ft এককে প্রকাশ হয়।

Relation between Chezy's C & Manning's n

$$C\sqrt{RS} = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\Rightarrow C R^{\frac{1}{2}} = \frac{1}{n} R^{2/3}$$

$$\Rightarrow C = \frac{1}{n} R^{\frac{2}{3}-\frac{1}{2}}$$

$$\Rightarrow \boxed{C = \frac{1}{n} R^{\frac{1}{6}}}$$

* for wide channel, $R = y$ (depth)

[যখন width, $b = 1$ unit ধরা হয় তখন wide channel বলে]

* সাধারণত discharge m^3/s কিন্তু wide channel এ discharge এর unit $m^3/s/m$ অথবা m^2/s

⇒ Flow Type in channel

Froude Number, $F_r = \dfrac{V}{\sqrt{g\frac{A}{T}}}$

* A = Area

T = Top width

For rectangular channel, $F_r = \frac{V}{\sqrt{gy}}$

For triangular channel, $F_r = \frac{V}{\sqrt{gy/2}}$

$F_r > 1 \rightarrow$ Supercritical flow

$F_r < 1 \rightarrow$ Subcritical flow

$F_r = 1 \rightarrow$ Critical flow

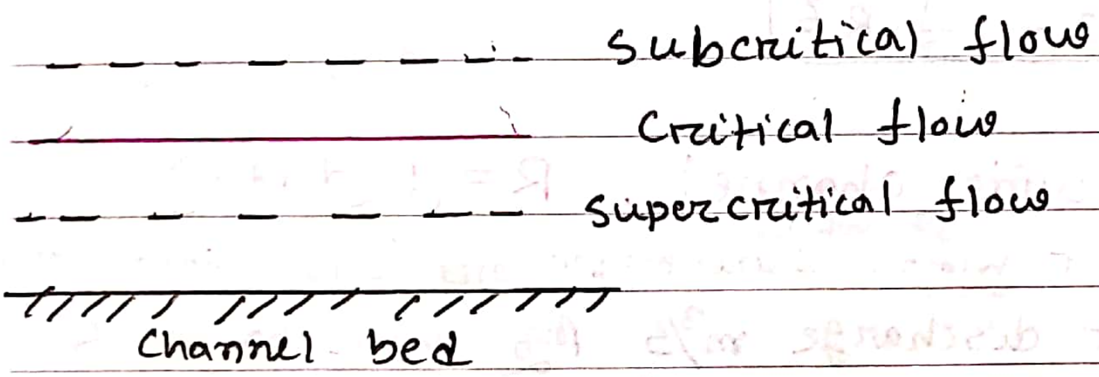
⇒ Velocity for critical flow in rectangular channel

$F_r = 1 = \frac{V}{\sqrt{gy_c}}$

$\therefore \boxed{V = \sqrt{gy_c}}$

Here,
$y = y_c$ = critical depth

⇒ Flow depth in different types of flow:



* Critical flow তে সবচেয়ে কম energy থাকে এবং সবচেয়ে বেশি discharge পাওয়া যায়।

* Supercritical ~~flow~~ flow তে যেহেতু depth সবচেয়ে কম হয় সেহেতু same discharge এর জন্য velocity সবচেয়ে বেশি পাওয়া যায়।

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⇒ Supercritical flow এর সামনে বাঁধ দিলে flow depth হঠাৎ বেড়ে যায়। একে hydraulic jump বলে। সামনে বাঁধ দিলে হেতু velocity কমে যায়, তাই discharge constant রাখতে flow depth বাড়ে supercritical থেকে subcritical হয়ে যায়।

⇒ Natural প্রায় সকল flow অর্থাৎ নদী, খালের flow হয় subcritical. এদের depth অনেক বেশি, velocity কম। ফলে সামনে বাঁধ দিলেও side কেটে চলে যায়, velocity এর পরিবর্তন হয় না।

Critical depth Calculation:

$$\boxed{\frac{Q^2}{g} = \frac{A^3}{T}}$$ ← universal formula for any section

T = Top width

For rectangular channel, $y_c = \left(\frac{q^2}{g}\right)^{1/3}$

* q = unit discharge $= \frac{Q}{\text{width}, B}$

For triangular channel,

$$y_c = \left(\frac{2Q^2}{gs^2}\right)^{1/5}$$

critical velocity for rectangular channel,

$v_c = \sqrt{gy_c}$

For triangular channel, ~~critical~~
critical velocity, $v_c = \sqrt{g y_c/2}$

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A wide Channel has $n = 0.026$ & $S_0 = 0.0028$. Calculate the normal depth & velocity of the flow if discharge is $30\ m^3/sec/m$. [BPDB'16]

Soln: $q = \frac{1}{n} A R^{2/3} S^{1/2}$

$\Rightarrow q = \frac{1}{n} y \cdot y^{2/3} \cdot S^{1/2}$

$\Rightarrow 30 = \frac{1}{0.026} y^{5/3} \sqrt{0.0028}$

$\therefore y = 5.025\,m$

velocity $= \frac{30}{5.025} = 5.97\ m/s$

Calculate normal depth of earthen channel if discharge is $10\,m^3/sec$ & channel slope is 1cm/km. Assume best triangular channel. [BIWTA 19]

take, $n = 0.025$

Soln: $10 = \frac{1}{0.025} y^2 \cdot \left(\frac{y}{2\sqrt{2}}\right)^{2/3} \cdot \sqrt{10^{-5}}$

$\Rightarrow y^{\frac{8}{3}} = \frac{10 \times 0.025 \times (2\sqrt{2})^{\frac{2}{3}}}{\sqrt{10^{-5}}}$

$\therefore y = 6.67\,m$

For best section,

$A = y^2$

$R = \frac{y}{2\sqrt{2}}$

$S = \frac{1cm}{km}$

$= \frac{10^{-5}km}{km}$

$= 10^{-5}$

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* A wide channel having longitudinal slope 0.001 . Manning's roughness coefficient 0.02 & velocity $1.6 ms^{-1}$. calculate normal depth .

Soln:
$$1.6 = \frac{1}{0.02} \times y^{2/3} \times \sqrt{0.001}$$
$$\therefore y = 1.02 m$$

* wide channel এ ~~depth~~ $R = y$
* যদি কোন ft এ থাকত তবে $\frac{1.486}{n}$ হতো। inch থাকলে হবে না। এজন্যই ft এ নিয়ে আসতে হবে।

* A wide channel discharge $3 m^3/sec$ and longitudinal slope 0.0025 and Manning's $n = 0.025$ Calculate the normal depth & velocity

[PGCB'20] [DPDC'20] [BIFPCL'21]

Soln:

$$3 = \frac{1}{0.025} \times (1 \times y) \times y^{2/3} \times \sqrt{0.0025}$$

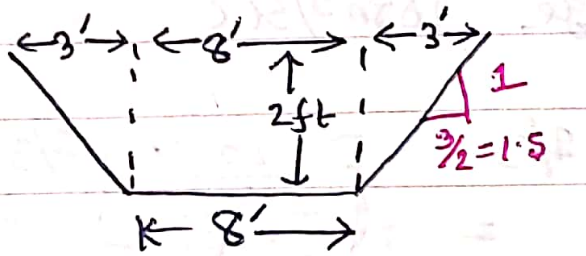
$$\therefore y^{\frac{5}{3}} = 1.5$$

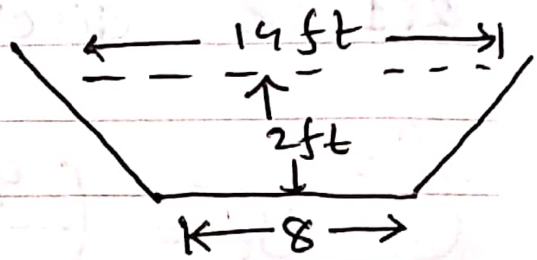
$$\therefore y = 1.275\,m$$

$$\text{velocity, } v = \frac{3}{1 \times 1.275} = 2.35\,m/s \text{ Ans}$$

\# The trapezoidal channel shown below has a Manning's coefficient of $n = 0.013$ & is laid at a slope of 0.0028, the depth of flow is 2ft. What is the flow rate? [SQFL'17]

Solⁿ:





$$\text{Area} = (b+sy)y$$
$$= (8+1.5\times2)\times2 = 22\,ft^2$$

$$\text{Perimeter}, P = 8 + 2\times2\sqrt{1+1.5^2} = 15.21\,m$$

$$\therefore R = \frac{22}{15.21} = 1.44\,m$$

$$\therefore Q = \frac{1.486}{\cancel{0.0028}\ 0.013} \times 22 \times (1.44)^{2/3} \times \sqrt{0.0028}$$
$$= 170\ ft^3/sec \quad \text{Ans}$$

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* Compute the critical depth & velocity in

(i) a rectangular channel with width of 6m & discharge $20 m^3/sec$

(ii) Triangular channel with side slope 2:1 & discharge $10 m^3/sec$

Soln:

(i) $y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left[\frac{\left(\frac{20}{6}\right)^2}{9.81}\right]^{1/3} = 1.04 m$

critical velocity, $V_c = \sqrt{g y_c} = \sqrt{9.81 \times 1.04} = 3.2 m/s$

(ii) $y_c = \left(\frac{2Q^2}{g s^2}\right)^{1/5} = \left(\frac{2 \times 10^2}{9.81 \times 2^2}\right)^{1/5} = 1.385 m$

critical velocity, $V_c = \sqrt{g y_c/2}$ ***

$= \sqrt{9.81 \times \frac{1.385}{2}} = 2.61 m/s$

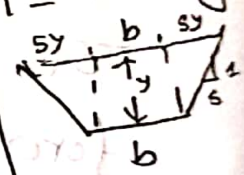
*** A ~~rectangular~~ Trapezoidal channel has a width of 4m & side slopes of 1.5 : 1 carries a certain discharge.

(a) If the critical depth is 1.7m, calculate the discharge in the channel

(b) If this discharge was observed to be flowing at a depth of 2.50m in a reach, estimate the froude number of the flow.

$A = (b+sy)y$

$T = b + 2sy$



Soln: We know,

$$\frac{Q^2}{g} = \frac{A^3}{T}$$

$$\Rightarrow \frac{Q^2}{9.81} = \frac{[(4+1.5\times1.7)1.7]^3}{4+2\times1.5\times1.7}$$

$$\Rightarrow Q = 38.58\ m^3/s$$

(b) $$V = \frac{Q}{A} = \frac{38.58}{(4+1.5\times2.5)\times2.5} = 1.99$$

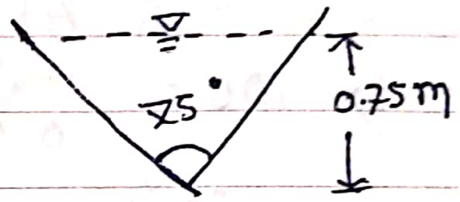
$$\frac{A}{T} = \frac{(4+1.5\times2.5)\times2.5}{4+2\times1.5\times2.5} = 1.68$$

$$Fr = \frac{V}{\sqrt{g\frac{A}{T}}} = \frac{1.99}{\sqrt{9.81\times1.68}} = 0.49$$

∴ subcritical flow

* A triangular channel with an apex angle of 75° carries a flow of $1.2 m^3/s$ at a depth of 0.75m. If the bed slope is 0.009, find the roughness coefficient (n) of the channel. What is the corresponding value of Chezy's coefficient (C). Also calculate velocity (v). [BWDB '22]

Soln:



$$A = y^2 \tan \theta/2$$
$$= (0.75)^2 \tan \frac{75}{2}$$
$$= 0.43 m^2$$

* $s = \tan \theta/2$

$$P = 2y\sqrt{1+\tan^2 \theta/2}$$
$$= 2 \times 0.75 \sqrt{1+\tan^2 75/2}$$
$$= 1.89 m$$

$$\therefore R = A/P = 0.227 m$$

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$
$$\Rightarrow 1.2 = \frac{1}{n} 0.43 \times (0.227)^{2/3} \times \sqrt{0.009}$$
$$\Rightarrow n = \frac{0.43 \times (0.227)^{2/3} \times \sqrt{0.009}}{1.2}$$
$$= 0.0126$$

$$V = \frac{Q}{A} = \frac{1.2}{0.43} = 2.8 m/s$$

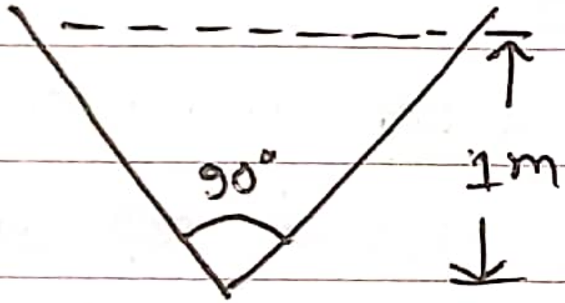
*** $V = C\sqrt{RS}$

$$\Rightarrow C = \frac{2.8}{\sqrt{0.227 \times 0.009}} = 61.94$$

Ans

* Determine the top width of best triangular channel section if flow depth is 1m [DMTCL'22]

Soln:



best section

Top width $= 2Sy$

$= 2 \times \tan\frac{90}{2} \times 1$

$= 2m$

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* A rectangular channel is to carry 100 cusec water. Calculate the dimensions of the best hydraulic x-section if the channel is lined with cement & is laid on a slope of 0.0001. Assume, $n = 0.012$

Solⁿ:

Cusec = $1 ft^3/sec$

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$\Rightarrow 100 = \frac{1.49}{0.012} \times (2y^2) \times \left(\frac{y}{2}\right)^{2/3} \times \sqrt{0.0001}$$

$$\Rightarrow y^{2+\frac{2}{3}} = 63.92$$

$$\Rightarrow y^{8/3} = 63.92$$

$$\therefore y = 4.75 \text{ ft}$$

For best rectangular section

$A = 2y^2$

$R = y/2$

$\therefore$ width, $B = 2y = 2 \times 4.75 = 9.5 ft$

* A cement lined rectangular channel 20ft wide carries water at a rate of $400 ft^3/sec$. Determine the slope required to maintain a depth of 4ft. Take, $n = 0.012$

$$\Rightarrow 400 = \frac{1.486}{0.012} \times (4 \times 20) \times \left(\frac{4 \times 20}{20 + 2 \times 4}\right)^{2/3} \times \sqrt{S}$$

$$\Rightarrow \sqrt{S} = 0.02$$

$$\therefore S = 0.0004$$

A long channel with a triangular cross section has a ~~side~~ side slope of 1:1 and a constant bed slope of 0.343°. What is the water depth when the channel carries 14cfts? Take, $n = 0.012$. Is the channel slope mild or steep?

Sol^n:

$$14 = \frac{1.486}{0.012} \times (1 \times y^2) \times \left(\frac{1 \times y^2}{2y\sqrt{1+1^2}}\right)^{2/3} \times (\tan 0.343)^{1/2}$$

$$\Rightarrow y^{2+\frac{2}{3}} = \frac{14 \times 0.012 \times (2\sqrt{2})^{2/3}}{1.486 \times \sqrt{\tan 0.343}} = 2.922$$

$$\therefore y^{\frac{8}{3}} = 2.922$$

$$\therefore y = 1.495 \text{ ft}$$

again $y_c = \left(\frac{2Q^2}{g s^2}\right)^{1/5} = \left(\frac{2 \times 14^2}{32.2 \times 1^2}\right)^{1/5} = 1.65 \text{ ft}$

$\because y < y_c$ $\therefore$ supercritical flow & slope is steep slope

Determine the dimensions of the most economical trapezoidal channel to carry 200 m³/s with a slope of 0.0004. Take $n = 0.016$

Sol^n:

For most economical Trapezoidal Channel,
$A = \sqrt{3}y^2$
$R = y/2$

$$200 = \frac{1}{0.016} \times (\sqrt{3}y^2) \times \left(\frac{y}{2}\right)^{2/3} \times \sqrt{0.0004}$$

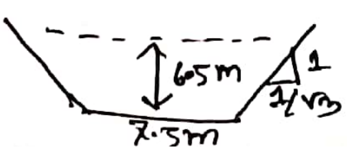
$$y^{\frac{8}{3}} = \frac{200 \times 0.016 \times 2^{2/3}}{\sqrt{3} \times \sqrt{0.0004}}$$

$$\therefore y^{8/3} = 146.64$$

$\therefore y = 6.5\text{m}$ $\quad\therefore$ depth; $y = 6.5\text{m}$

bottom width $= \frac{2y}{\sqrt{3}} = \frac{2 \times 6.5}{\sqrt{3}} = 7.5\text{m}$

side slope $= \frac{1}{\sqrt{3}} : 1$



Energy head in Open Channel Flow:

প্রতিটির Section এ total energy head same থাকে,

$$\underset{\text{depth}}{y} + \frac{v^2}{2g} + (\text{elevation যদি থাকে অথবা hump})$$

critical depth যদি হয় তবে, energy সবচেয়ে কম প্রাপ্ত হয়, তখন total energy head

$E_{min} = 1.5 y_c$ [rectangular channel]

$= 1.25 y_c$ [Triangular channel]

Hydraulic Jump:

Flow যদি super critical হয় তখন সামনের দিকে গেলে flow jump করে subcritical হয়ে যাবে।

লাফ দেওয়ার আগের ও পরের এ দুটি depth কে sequent depth বলে।

$$y_2 = \frac{y_1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

Jump height $= y_2 - y_1$

Energy loss in jump $= \dfrac{(y_2-y_1)^3}{4y_1y_2}$

* 4m wide rectangular channel with a sluice gate has an u/s water depth of 2.5m and down stream water depth of 0.4m. Calculate discharge through the channel. Neglect all losses [RPGCL '22]

Soln: $y_1 + \frac{v_1^2}{2g} = y_2 + \frac{v_2^2}{2g}$

$$\Rightarrow 2.5 + \frac{Q^2}{2g*4^2*2.5^2} = 0.4 + \frac{Q^2}{2g*4^2*0.4^2}$$

$$\Rightarrow \frac{Q^2}{2g*16} * \left(\frac{1}{0.4^2} - \frac{1}{2.5^2} \right) = 2.5 - 0.4$$

$$\Rightarrow \frac{Q^2}{313.92} = 0.34483$$

$$\Rightarrow Q = 10.4 m^3/s$$

A cement lined rectangular channel 20ft wide carries water at a rate of 400ft^3/sec. Calculate the critical height of a rise for depth 4ft.

Soln: critical depth, $y_c = \left[\frac{\left(\frac{400}{20}\right)^2}{32.2}\right]^{1/3}$

$= 2.32 ft$

$\therefore$ Energy head at critical depth $= 1.5 \times 2.32$
$= 3.47 ft$

Energy head at 4ft depth $= y + \frac{v^2}{2g}$

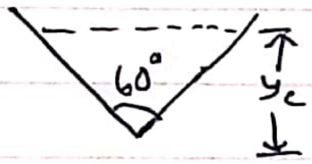
$= 4 + \frac{\left(\frac{400}{20\times4}\right)^2}{2\times32.2}$

$= 4.39 ft$

$\therefore$ ~~rise = 4.39 - 3(3.47 - 4.39) ft~~

rise $= (4.39 - 3.47)$ ft
$= 0.92 ft$ Ans

* Compute the critical depth for flow at 0.4 m^3/s through the x-section of below figure. If the normal depth is 1.2m, wheather the flow is subcritical or supercritical?



Soln: $y_c = \left(\frac{2Q^2}{gs^2}\right)^{1/5}$; $s = \tan\theta/2 = \tan 30$

$\therefore y_c = \left(\frac{2\times0.4^2}{9.81\times\tan^2 30}\right)^{1/5}$

$= 0.628 m$

Since normal depth $= 1.2m > 0.628m$ $\therefore$ subcritical

Date :　　　U/S → upstream
d/S → downstream

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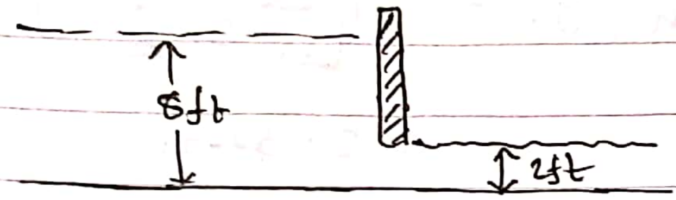
The U/S & d/S depth of a sluice gate are 8ft & 2ft respectively. The channel is of rectangular shape & 10ft wide. Find the discharge under the gate.



$$y_1 + \frac{v_1^2}{2g} = y_2 + \frac{v_2^2}{2g}$$

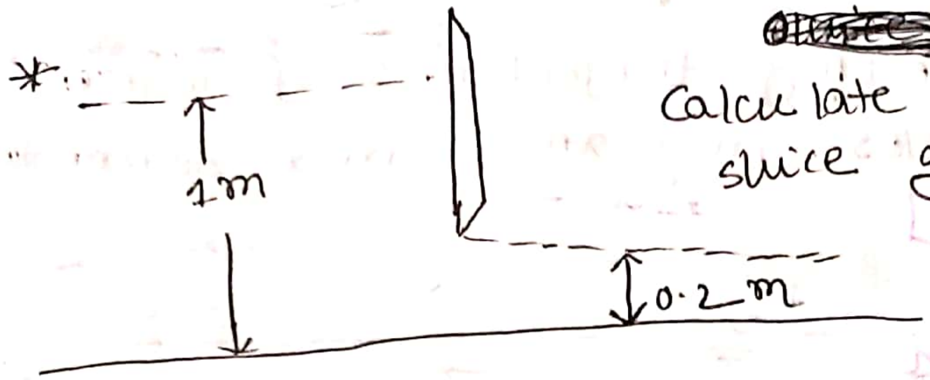
$$\Rightarrow 8 + \frac{Q^2}{2g b^2 y_1^2} = 2 + \frac{Q^2}{2g b^2 y_2^2}$$

$$\Rightarrow 6 = \frac{Q^2}{2g b^2}\left(\frac{1}{y_2^2} - \frac{1}{y_1^2}\right)$$

$$\Rightarrow Q = \sqrt{\frac{2 \times 32.2 \times 10^2 \times 6}{\frac{1}{2^2} - \frac{1}{8^2}}}$$

$$\therefore Q = 406.03 \ ft^3/sec$$

*



Calculate discharge of this sluice gate.

[DNCC'22 MIST]

$Q = \frac{V}{A}$

$= \frac{V}{by}$

$= \frac{V}{1 \cdot y}$

$= \frac{V}{y}$

Solⁿ: $y_1 + \frac{v_1^2}{2g} = y_2 + \frac{v_2^2}{2g}$

$\Rightarrow y_1 + \frac{Q^2}{2g y_1^2} = y_2 + \frac{Q^2}{2g y_2^2}$

$\Rightarrow 1 + \frac{Q^2}{2g \cdot 1^2} = 0.2 + \frac{Q^2}{2g \cdot (0.2)^2}$

$\Rightarrow \frac{Q^2}{2 \times 9.81}\left(\frac{1}{0.2^2} - \frac{1}{1^2}\right) = 1 - 0.2$

$\Rightarrow Q^2 = \frac{2 \times 9.81 \times 0.8}{\frac{1}{0.2^2} - \frac{1}{1^2}}$

$\therefore Q = \sqrt{\frac{2 \times 9.81 \times 0.8}{\frac{1}{0.2^2} - \frac{1}{1^2}}}$

$= 0.81 \; m^3/s/m$ width

A

Sub :

Day

Time : Date : / /

* A typical venturi flume of rectangular cross section has width of 2.5m at its upstream and 0.8m at its throat. For a perticular flow through the flume, the upstream depth is measured as 1.2m. If the theoritical discharge is 1.73 m³/sec, calculate the d/s water depth at the throat of the flume. ~~under~~ sub [RRI '21]

Soln:

$$y_1 + \frac{Q^2}{2gA_1^2} = y_2 + \frac{Q^2}{2gA_2^2}$$

$$\Rightarrow 1.2 + \frac{(1.73)^2}{2\times 9.81\times(2.5\times 1.2)^2} = y_2 + \frac{(1.73)^2}{2\times 9.81\times(0.8y_2)^2}$$

$$\Rightarrow 1.217 = y_2 + \frac{0.24}{y_2^2}$$

$$\Rightarrow y_2^3 - 1.217y_2^2 + 0.24 = 0$$

Solving the eqn, $y_2 = 0.952\text{m}$ Ans

In a rectangular channel, the discharge is $40 ft^3/sec$ per ft width. If a hydraulic jump occurs at a depth of 3ft, calculate the height of the jump and the energy dissipated per foot width.

Soln:

$$F_1 = \frac{V}{\sqrt{gy}} = \frac{\frac{40}{3}}{\sqrt{32.2 \times 3}} = 1.35$$

$$\therefore y_2 = \frac{y_1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

$$= \frac{3}{2}\left[\sqrt{1+8\times(1.35)^2} - 1\right]$$

$$= 4.42 ft$$

[BWDB'16]

$\therefore$ Jump Height $= (4.42-3) ft = 1.42 ft$ ***

$\therefore$ Energy head loss in jump, $\Delta H = \frac{(y_2-y_1)^3}{4y_1y_2}$

$$= \frac{(1.42)^3}{4\times 4.42\times 3}$$

$$= 0.054$$

$\therefore$ Energy dissipated $= \gamma Q \Delta H$

$$= 62.43 \times 40 \times 0.054$$

$$= 134.85 \text{ lb-ft/sec}$$

Date: / /

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* Water flows in a horizontal rectangular channel 6m wide at a depth of 0.52m and a velocity of 15.2m/s. If a hydraulic jump forms in the channel, determine the power dissipation in the jump in HP unit.

Soln: $Fr_1 = \dfrac{15.2}{\sqrt{9.81 \times 0.52}} = 6.73\,m$

$$y_2 = \frac{y_1}{2}\left[\sqrt{1+8F_1^2} - 1\right]$$

$$= \frac{0.52}{2}\left[\sqrt{1+8\times(6.73)^2} - 1\right]$$

$$= 4.7\,m$$

Energy head loss, $\Delta H = \dfrac{(4.7-0.52)^3}{4\times 4.7\times 0.52}$

$$= 7.47\,m$$

Power dissipation $= \dfrac{\gamma Q \Delta H}{746}$ KW → W

$$= \frac{9.81\times 10^3 \times (6\times 0.52\times 15.2) \times 7.47}{746}$$

$$= 4658.53\ HP$$

Date: / /

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A hydraulic jump takes place in a 0.4m wide laboratory flume. The initial depth of the jump is 20mm and the flow rate is 31 l/sec. The flume is horizontal, rectangular & smooth. Calculate the sequent depth of the jump. If the dissipated power could be transformed into electricity, how many 40W bulbs could be lighted?

[BGDCL'21]

Solⁿ: $F_1 = \dfrac{\dfrac{31\times10^{-3}}{0.4\times20\times10^{-3}}}{\sqrt{9.81\times20\times10^{-3}}} = 8.75$

$\therefore y_2 = \dfrac{20\times10^{-3}}{2}\left[\sqrt{1+8\times(8.75)^2} - 1\right]$

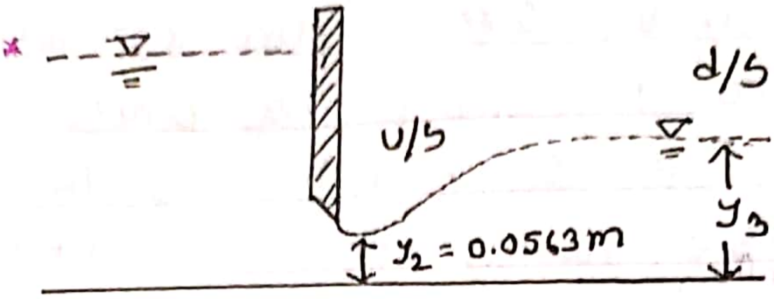
$= 0.237\text{ m}$

$\therefore$ head loss $= \dfrac{(0.237-0.02)^3}{4\times0.237\times0.02} = 0.54\text{m}$

$\therefore$ Power dissipated $= 9.81\times10^3 \times 31\times10^{-3} \times 0.54$

$= 164.22\text{ Watt}$

No of bulbs $= \dfrac{164.22}{40} \approx 4$ Nos

* 

Determine the depth of flow downstream of the jump if the velocity under the gate is 5.33m/s

Solⁿ: $Fr_2 = \dfrac{5.33}{\sqrt{9.81 \times 0.0563}} = 7.17$

$$y_3 = \frac{y_2}{2}\left[\sqrt{1+8Fr_2^2} - 1\right]$$

$$= \frac{0.0563}{2}\left[\sqrt{1+8\times(7.17)^2} - 1\right]$$

$$= 0.54m$$ Ans:

* Water flows from a lake into a steep rectangular channel of 3m wide. Determine the lake level above the channel bed at the outfall if discharge is $40m^3/sec$.

Solⁿ: lake থেকে rectangular channel এ পানি প্রবেশ-মুহূর্তে পানির level critical depth এ থাকে।

$\therefore$ lake level at the outfall, $y_c = \left[\dfrac{\left(\frac{40}{3}\right)^2}{9.81}\right]^{1/3}$

$$= 2.627m$$

* A rectangular channel 2m wide carries 2.2m³/sec of water in subcritical uniform flow at a depth of 1m. What is the lowest transverse hump in the bottom such that critical depth is attained at the peak?

Solⁿ:

$$y_1 + \frac{v^2}{2g} = E_{min} + z$$

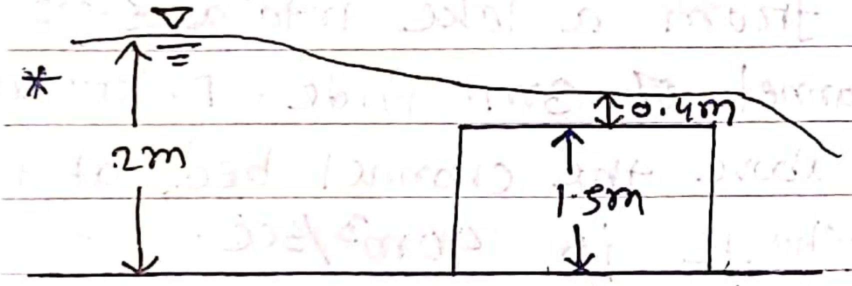
$E_{min} \rightarrow 1.5 \times y_c$

critical depth থাকলে তবে minimum head হয়।

$$\Rightarrow 1 + \frac{\left(\frac{2.2}{2\times1}\right)^2}{2\times9.81} = 1.5 \times \left[\frac{\left(\frac{2.2}{2}\right)^2}{9.81}\right]^{1/3} + z$$

$$\Rightarrow z = 0.315\,m \quad \text{A}$$

* (B) What is the upstream velocity?



broad crested weir

Solⁿ: যেহেতু width দেওয়া নাই, তাই unit width ধরতে হবে।

$$y_1 + \frac{v_1^2}{2g} = y_2 + \frac{v_2^2}{2g} + z$$

$$\Rightarrow 2 + \frac{q^2}{2g\times2^2} = 0.4 + \frac{q^2}{2g\times0.4^2} + 1.5$$

$$\Rightarrow \frac{q^2}{2g}\left(\frac{1}{0.16}-\frac{1}{4}\right) = 2-0.4-1.5 = 0.1$$

$$\Rightarrow q = \sqrt{\frac{0.1\times 2\times 9.81}{\frac{1}{0.16}-\frac{1}{4}}} = 0.572\ m^3/s/m$$

$$\therefore \text{upstream velocity} = \frac{0.572}{2} = 0.286\ m/s$$

A

Shortcut

* unit discharge in a channel, $q = \sqrt{\frac{2g\Delta H}{\frac{1}{y_2^2}-\frac{1}{y_1^2}}}$ $\therefore$ Discharge, $Q = q\times B$

$V = \frac{\text{discharge}}{\text{Area}}$

* water flows at a velocity of 1m/s and a depth of 1.5m in a long rectangular channel of 3m width. Compute the height of a smooth upward step in the channel bed to produce critical flow and the change in water level [BWDB'19] [JB'17]

Soln: $E = E_{critical} + \Delta Z$. $\boxed{q = v\times y = 1.5\times 1}$ ***

$$\Rightarrow y_1 + \frac{v_1^2}{2g} = 1.5y_c + \Delta Z$$

$$\Rightarrow 1.5 + \frac{1^2}{2\times 9.81} = 1.5\times\left[\frac{(1\times 1.5)^2}{9.81}\right]^{1/3} + \Delta Z$$

$$\Rightarrow \Delta Z = 0.63m$$

* $y_c = \left[\frac{(1\times 1.5)^2}{9.81}\right]^{1/3} = 0.612$ [এটা আলাদাভাবে উঠিয়ে দিলাম।]

Change in water level, $\Delta H = (1.5 - 0.612 - 0.63) = 0.258\ m$

W-1

* A rectangular channel has a width of 2m and carries a discharge of 4.8 m³/sec with a depth of 1.6m. At a certain cross-section, a small smooth hump with a flat top and a height 0.10m is proposed to be built. Calculate the likely change in the water surface. Neglect the energy loss.

Sol^n: Energy head at u/s,

$$E_1 = 1.6 + \frac{\left(\frac{4.8}{2\times1.6}\right)^2}{2\times9.81} = 1.715\,m$$

at section 2, $E_2 = 1.715 - 0.10 = 1.615$

$$\therefore E_2 = 1.615 = y_2 + \frac{\left(\frac{4.8}{2\times y_2}\right)^2}{2\times9.81}$$

$$\Rightarrow 1.615 = y_2 + \frac{0.2935}{y_2^2}$$

$$\Rightarrow 1.615 y_2^2 = y_2^3 + 0.2935$$

Solving, $y_2 = 1.48\,m$ or $y_2 = 0.517\,m$

but $y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left[\frac{\left(\frac{4.8}{2}\right)^2}{9.81}\right]^{1/3} = 0.837\,m$

$\therefore$ minimum specific energy head, $E_c = 1.5\times0.837 = 1.256$

$\because E_2 > E_c = 1.256$

$\therefore$ The flow must be subcritical, hence, $y_2 = 1.48\,m > 0.837\,m$

$\therefore$ The drop at water surface elevation is,

$$\Delta y = 1.6 - (1.48 + 0.10) = 0.02m$$ A

* A rectangular channel 2.5m wide carries $6m^3/sec$ of flow at a depth of 0.50m. calculate the height of a flat topped hump required to be placed at a section to cause critical flow. The ~~obstruction~~ obstruction energy loss due to the obstruction can be taken as 0.1 times by the hump the upstream velocity head.

Soln:

$$v_1 = \frac{6}{2.5 \times 0.5} = 4.8m/s$$

$$E_1 = 0.5 + \frac{(4.8)^2}{2 \times 9.81} = 0.5 + 1.1743 = 1.6743m$$

depth at d/s

$$y_c = \left[\frac{\left(\frac{6}{2.5}\right)^2}{9.81}\right]^{1/3} = 0.837m$$

$$\therefore E_2 = 1.5 \times 0.837 = 1.256m$$

$$\therefore E_1 = E_2 + \Delta z + H_L$$

$$1.6743 = 1.256 + \Delta z + 0.1 \times 1.1743$$

$$\Rightarrow \Delta z = 0.3m$$ A

Date :
Sun Mon Tue Wed Thu Fri Sat

* Water flows at a velocity of 1m/s and a depth of 1.5m in a long rectangular channel 3m wide. Compute the height of a smooth upward step in the channel bed to produce critical flow & the change in water level.

$$y_c = \left(\frac{Q^2}{gB^2}\right)^{1/3}$$

Soln:

$$y_c = \left(\frac{4.5^2}{9.81 \times 3^2}\right)^{1/3} = 0.61m$$

$$1.5 + \frac{1^2}{2 \times 9.81} = 1.5 \times 0.61 + \Delta z$$

$\therefore \Delta z = 0.635m$

$\therefore$ hump height $= 0.635m$

Change in water level $= 1.5 - 0.61 - 0.635$
$= 0.255m$.

* A trapezoidal channel has a bottom width of 6m & side slope 1:1. The depth of flow is 1.5m at a discharge of $15m^3/s$. Determine specific energy in terms of head of water [BUET'12]

Soln: $A = (b+sy)y = (6+1\times1.5)1.5 = 11.25m^2$

$\therefore$ Energy $= 1.5 + \dfrac{15^2}{2\times9.81\times(11.25)^2} = 1.59m$

$$\boxed{y + \frac{Q^2}{2gA^2}}$$

*Depth of water flowing in a 3m wide rectangular channel is 2m. The channel carries a discharge of $12 m^3/s$. Calculate the minimum bed width at contraction which just causes the critical flow without ~~chain~~ changing the upstream water level.

Sol^n: Specific energy at section 1,

$$E_1 = y_1 + \frac{v_1^2}{2g}$$

$$= 2 + \frac{\left(\frac{12}{3\times2}\right)^2}{2\times9.81} = 2.2038 m$$

at section 2, depth is critical,

$$\therefore E_c = \frac{3}{2} y_c = 2.2038$$

$$\Rightarrow y_c = \frac{2}{3}\times2.2038 = 1.47 m$$

again, $\Rightarrow \left(\frac{q_2^2}{g}\right)^{1/3} = 1.47$

$$\Rightarrow \frac{q_2^2}{g} = 3.17$$

$$\Rightarrow q_2^2 = 31.114$$

$$\Rightarrow q = 5.578$$

$$\Rightarrow \frac{Q}{B_2(min)} = 5.578$$

$$\Rightarrow B_2(min) = \frac{Q}{5.578} = \frac{12}{5.578} = 2.15 m$$

A

WI * A rectangular channel is 3.50m wide conveys a discharge of $15 m^3/sec$ at a depth of 2m. It is proposed to reduce the width of the channel at a hydraulic structure. Determine the minimum width which does not cause choking.

Soln: Specific energy at U/S

$$E_1 = 2 + \frac{\left(\frac{15}{3.5\times 2}\right)^2}{2\times 9.81} = 2.234 m$$

for no chocking condition,
~~Maximum depth which will not cause~~

~~Choking is~~ $y_c = \frac{2}{3} E$ $\quad [E_c = \frac{3}{2} y_c]$

$$= \frac{2}{3}\times 2.234$$

$$= 1.489 m$$

again, $y_c = \left[\frac{\left(\frac{Q}{B}\right)^2}{g}\right]^{1/3}$

$$\Rightarrow (1.489)^3 = \frac{\frac{15^2}{B^2}}{9.81}$$

$$\Rightarrow \therefore B_{min} = 2.635 m$$

$\therefore$ minimum width $= 2.635m$

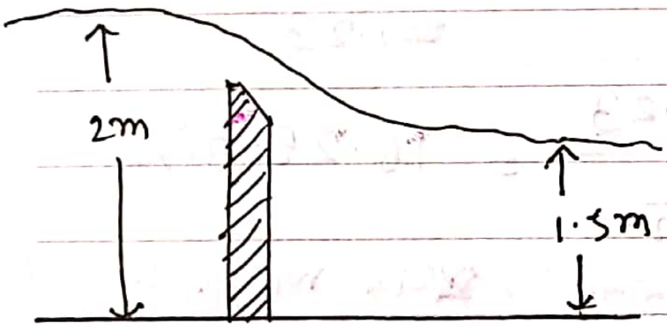
Force acting on hydraulic wall structure

$$F = \frac{1}{2} b \gamma (y_1^2 - y_2^2) - \rho Q \Delta V$$

* ΔV ~~স~~বসময়ই positive হতে হবে ।
* $\gamma = 9.81 \times 10^3 \, N/m^3$ এ নিয়ে যেতে হবে ।
  $= 62.43 \, lb/ft^3$

water density, $\rho = 1000 \, kg/m^3$
$= 1.94 \, slugs/ft^3$ ***

$$\rho = \frac{\gamma}{g} = \frac{62.43}{32.2} = 1.94$$



If unit discharge is $4 \, m^2/s$. Estimate energy loss & force on weir.

Sol^n: $v_1 = \frac{4}{2} = 2 \, m/s$, $v_2 = \frac{4}{1.5} \doteq 2.67 \, m/s$

Applying energy eq^n,

$$2 + \frac{2^2}{2 \times 9.81} = 1.5 + \frac{(2.67)^2}{2 \times 9.81} + h_L$$

$$\Rightarrow h_L = 0.34 \, m$$

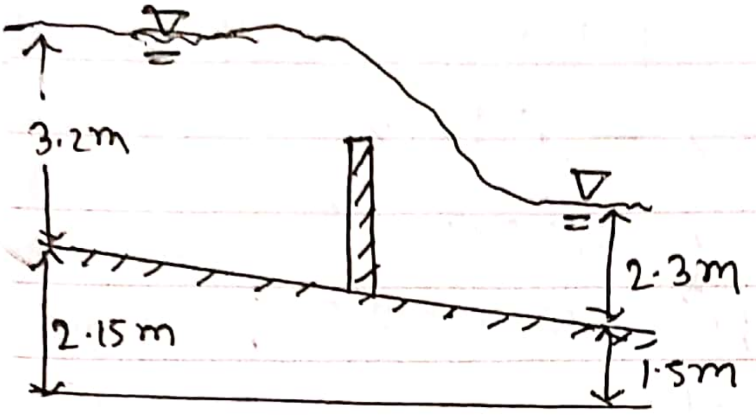
$$\text{Force} = \frac{1}{2} \gamma (y_1^2 - y_2^2) - \rho q \Delta v$$

$$= \frac{1}{2} \times 9.81 \times 1000 \times (2^2 - 1.5^2) - 1000 \times 4 \times (2.67 - 2)$$

$$= 5903.25 \, N \quad \underline{A}$$

$\Delta H = 3.2 + 2.15 - 2.3 - 1.5 - \underset{HL}{0.32} = 1.23^m$

$$q = \sqrt{\frac{2g\Delta H}{\frac{1}{y_2^2} - \frac{1}{y_1^2}}} = \sqrt{\frac{2\times 9.81\times 1.23}{\frac{1}{2.3^2} - \frac{1}{3.2^2}}} = 16.25\ m^2/s$$



Width of the channel 5m, head loss 0.32m. Calculate the force acting on the wall [SGEL'20]

Soln:

$$2.15 + 3.2 + \frac{Q^2}{2\times g\times 3.2^2} = 2.3 + 1.5 + \frac{Q^2}{2g\times 2.3^2} + 0.32$$

$$\Rightarrow \frac{q^2}{2g}\left(\frac{1}{2.3^2} - \frac{1}{3.2^2}\right) = 2.15 + 3.2 - 2.3 - 1.5 - 0.32$$
$$= 1.23$$

direct short cut

$$\Rightarrow q = \sqrt{\frac{2\times 9.81\times 1.23}{\frac{1}{2.3^2} - \frac{1}{3.2^2}}} = 16.25\ m^2/s$$

এখানে eqn সহজ করার জন্য প্রথমে width consider করিনি।

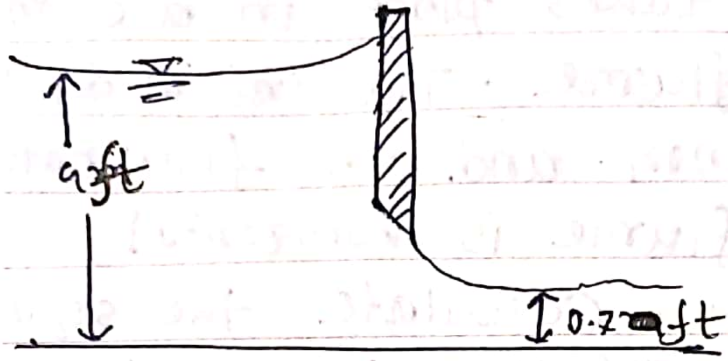
$$\therefore Q = qB = 16.25\times 5 = 81.25\ m^3/s$$

$$\therefore F = \frac{1}{2}\gamma B(y_2^2 - y_1^2) - \rho Q\Delta V$$

$$= \frac{1}{2}\times 9.81\times 10^3\times 5\,(3.2^2 - 2.3^2) - 10^3\times 81.25\times\left(\underbrace{\frac{81.25}{2.3\times 5}}_{V_2} - \underbrace{\frac{81.25}{3.2\times 5}}_{V_1}\right)$$

$$= -40052.51\ N$$

$$= -40.0525\ kN$$



compute the force per unit width

~~Solⁿ: $q = \sqrt{2\times 9.81\times(4-0.}$~~

Solⁿ:

$$q = \sqrt{\frac{2\times 32.2\times(4-0.7)}{\frac{1}{0.7^2}-\frac{1}{4^2}}} = 10.36\ ft^2/sec$$

$$\therefore v_1 = \frac{10.36}{4} = 2.6 ft/sec \ ; v_2 = 14.8 ft/sec$$

$$\therefore F = \frac{1}{2}\gamma(y_1^2 - y_2^2) - \rho q \Delta v$$

$$= \frac{1}{2}\times 62.43(4^2 - 0.7^2) - \frac{62.43}{32.2}\times 10.36\times(14.8-2.6)$$

$$= 238.4\ lb\text{-}ft/sec/m\ width$$

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Calculate the rate of flow over a sharp crested rectangular weir 6ft wide & 3ft height, If the head over the weir 10 in & $C_d = 0.63$

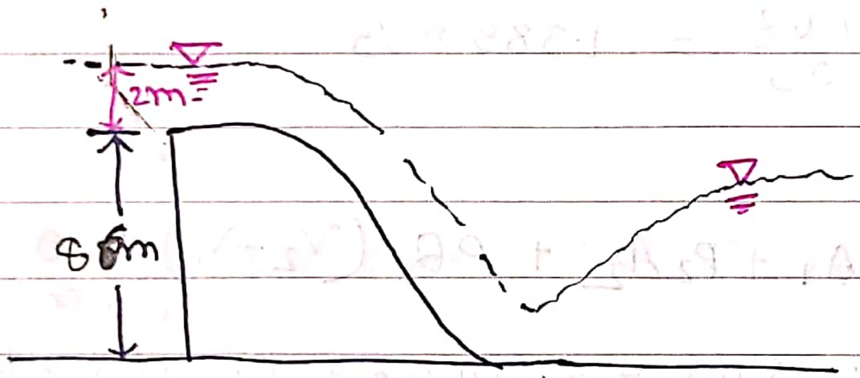
height 3ft কোনো কাজে নাই

Soln:
$$Q = \frac{2}{3} C_d A V \qquad A = B \times head$$

$$= \frac{2}{3} \times 0.63 \times \left(6 \times \frac{10}{12}\right) \times \sqrt{2 \times 32.2 \times \frac{10}{12}}$$

$$= 15.4 \, ft^3/sec$$

#



The length of spillway is 100m. $C_d = 2.4$ Compute discharge.

Soln:
$$Q = \frac{2}{3} C_d (b \times h) \times \sqrt{2gh}$$

$$= \frac{2}{3} \times 2.4 \times (100 \times 2) \times \sqrt{2 \times 9.81 \times 2}$$

$$= 2004.54 \, m^3/sec$$

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* The head regulator of a canal has three openings each 3m wide. The water is flowing between the upper and lower gates. The vertical openings of the gate is W = 1.0m. The head on the regulator is 0.45m. If the upstream water level rises by 0.2m, how much the upper gates must be lowered to maintain the canal discharge unaltered. Use, $C_d = 0.62$.
[MSc WRE '19]

Soln: total width of regulator openings $= 3 \times 3$
$b = 9m$

discharge, $Q = C_d \cdot b \cdot w \cdot \sqrt{2gy_1}$

$$= 0.62 * 9 * 1 * \sqrt{2 * 9.81 * 0.45} \quad \text{--- (i)}$$

In second case, when U/S water level rises by 0.20m, let the gate opening be xm to keep the discharge unaltered.

$$\therefore Q = 0.62 * 9 * x * \sqrt{2 * 9.81 * (0.45 + 0.20)} \quad \text{--- (ii)}$$

$\therefore$ from eqn (i) & (ii),

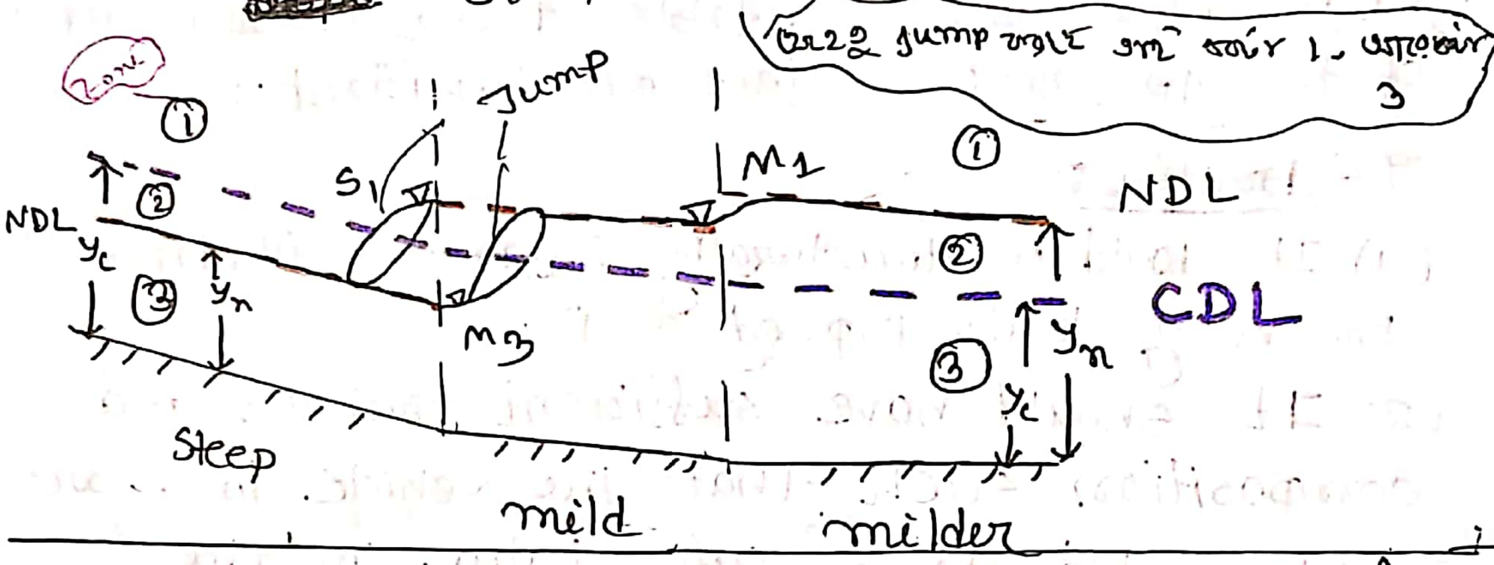
$$x * \sqrt{0.65} = 1 * \sqrt{0.45}$$

$$\Rightarrow x = 0.83m$$

Hence, the get must be lowered by an amount of $(1 - 0.83)m = 0.17m$ Ans

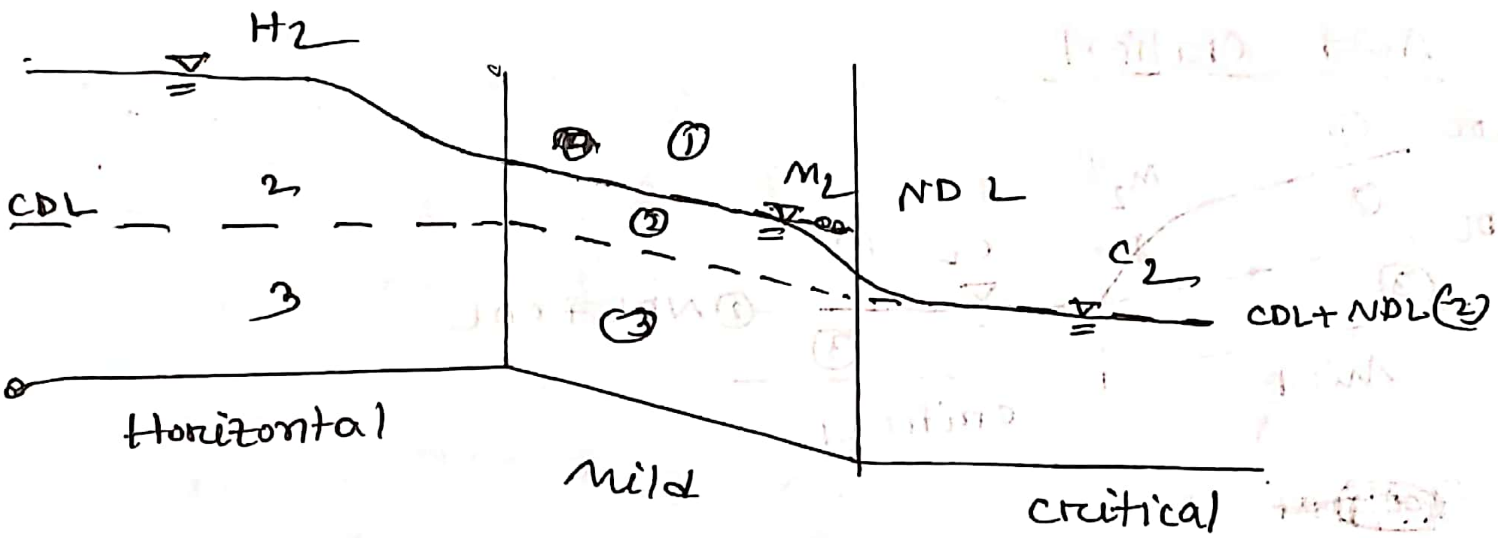
* ✱✱✱ Draw water surface profile for the following channel slope — [M.GMC'22]

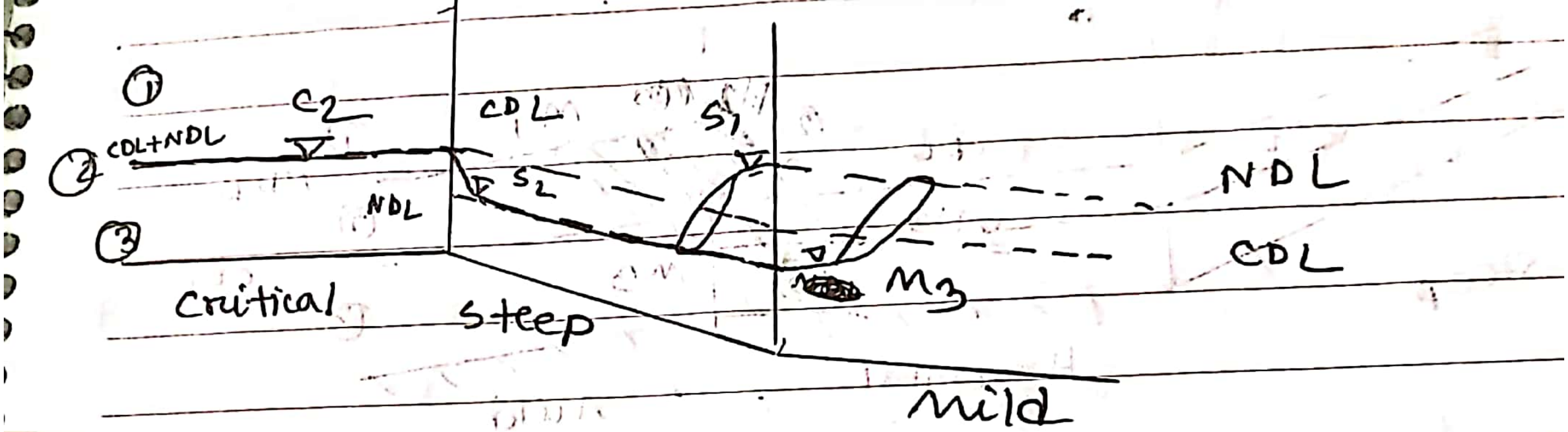
~~slope~~ Steep – Mild – Milder

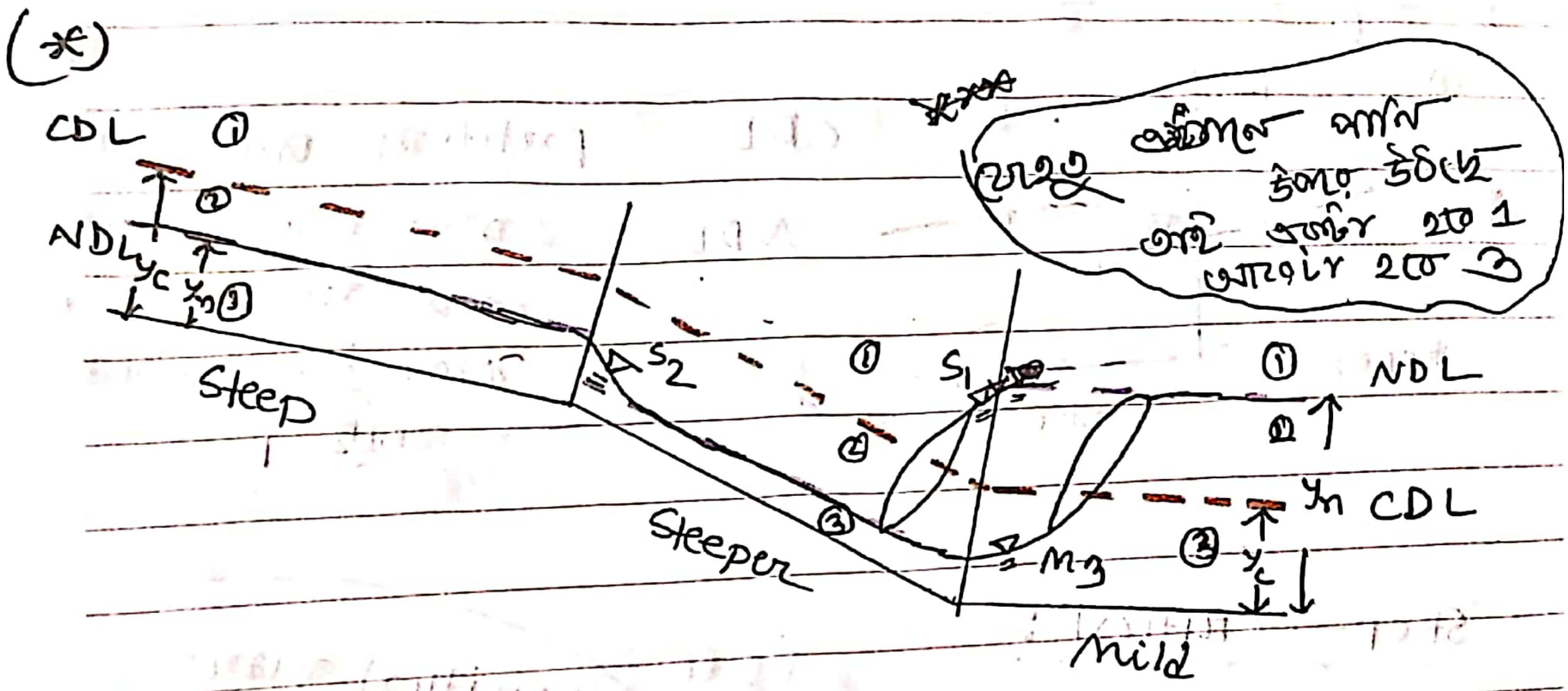


* এক NDL থেকে আরেক NDL এ যাওয়ার সময় যদি CDL cross করে তবে jump হবে।

* Zone 1, 3 থেকে সবসময় উপরে উঠে, Zone (2) থেকে সবসময় নিচে নামে।




1
2 CDL+NDL
3
C2
CDL
NDL
S1
S2
NDL
CDL
M3
Critical
Steep
Mild


(*)
CDL
1
2
3
NDL
y_c
y_n
S_2
Steep
Steeper
1
2
3
S_1
M_3
NDL
y_n CDL
y_c
Mild
যেহেতু
পানি
উপরে উঠছে
তাই প্রথমে হবে 1
আবার হবে 3

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(9) A 1m wide rectangular channel has a bed slope of 0.0016 and the Manning's coefficient is 0.04. Uniform flow takes place in the channel at a flow depth of 0.5m. Determine slope type.

[DMTCL Line 55]

Soln:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2}$$

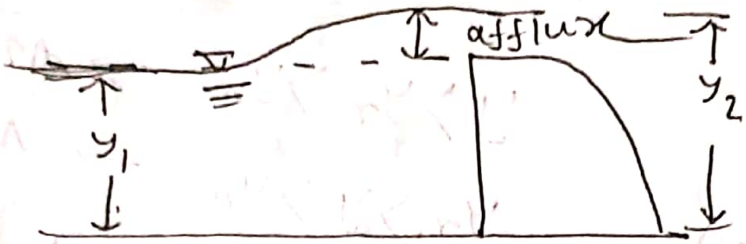
$$= \frac{1}{0.04} \times (1 \times 0.5) \times \left(\frac{1 \times 0.5}{1+2\times 0.5}\right)^{2/3} \times \sqrt{0.0016}$$

$$= 0.1984 \ m^3/s$$

$$\therefore y_c = \left(\frac{Q^2}{gB^2}\right)^{1/3} = \left[\frac{(0.1984)^2}{9.81 \times 1^2}\right]^{1/3} = 0.16m$$

$$\because y_n = 0.5 > y_c = 0.16$$

$\therefore$ Mild slope Ans



Length of back water curve, $l = \frac{E_1 - E_2}{S_b - S_e}$

S_b = given bed slope
S_e = energy line slope
E_1 = energy head at y_1
E_2 = energy head at y_2

** A rectangular channel 8m wide, bed slope of 0.0016 & n = 0.0025. Normal & critical depth are 1m & 0.526m. Compute the length of back water curve created by a dam which backs up a depth of 2m immediately behind the dam in a gradually varied flow. [BWDB'19]

$$v_1 = \frac{1}{0.0025} \times \left(\frac{8\times1}{8+2\times1}\right)^{2/3} \times (0.0016)^{1/2}$$

$$= 13.8\ m/s$$

$$E_1 = 1 + \frac{(13.8)^2}{2\times9.81} = 10.71\ m/s$$

(width = 8 m = const)

$$V_2 = \frac{V_1 y_1}{y_2} = \frac{13.8 \times 1}{2} = 6.9 \ m/s$$

$$E_2 = 2 + \frac{(6.9)^2}{2 \times 9.81} = 4.43 \ m$$

$\boxed{\text{now, } S_e \text{ calculation}}$

$$y_{av} = \frac{1+2}{2} = 1.5 \ m$$

$$V_{av} = \frac{V_1 y_1}{y_{av}} = \frac{13.8 \times 1}{1.5} = 9.2 \ m/s$$

এখানে $V_{av} = \frac{v_1 + v_2}{2}$ হবেনা কারণ V_{av} হবে avg depth এর জন্য velocity

$$R_{av} = \frac{8 \times 1.5}{8 + 2 \times 1.5} = 1.091 \ m$$

$$V_{av} = \frac{1}{n} R_{av}^{2/3} S_f^{1/2}$$

$$\Rightarrow 9.2 = \frac{1}{0.0025} \times (1.091)^{2/3} \times \sqrt{S_f}$$

$$S_f = 0.00047$$

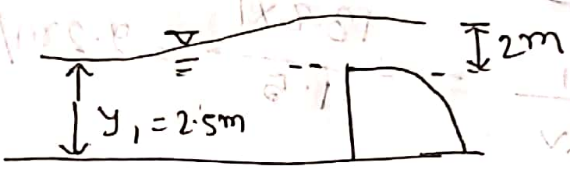
∴ length of backwater curve, $L = \frac{E_1 - E_2}{S_0 - S_f}$

$$= \frac{10.71 - 4.43}{0.0016 - 0.00047}$$

$$= 5.557 \ km \ \underline{\text{A.S}}$$

* Determine the length of backwater curve caused by an afflux of 2m in a rectangular channel of width 40m & depth 2.5m. The slope of bed is given as 1 in 11000. Take manning's $n = 0.03$

Solⁿ:



$\therefore y_2 = 2.5 + 2$
$= 4.5m$

$$v_1 = \frac{1}{0.03}\left(\frac{40 \times 2.5}{40 + 2 \times 2.5}\right)^{2/3} \times \sqrt{\frac{1}{11000}} = 0.54 \, m/s$$

$$E_1 = 2.5 + \frac{(0.54)^2}{2 \times 9.81} = 2.51 \, m$$

$$v_2 = \frac{0.54 \times 2.5}{4.5} = 0.3$$

$$\therefore E_2 = 4.5 + \frac{(0.3)^2}{2 \times 9.81} = 4.5 \, m$$

$$y_{avg} = \frac{2.5 + 4.5}{2} = 3.5$$

$$\therefore V_{avg} = \frac{V_2 y_2}{y_{avg}} = \frac{0.3 \times 4.5}{3.5} = 0.385 \, m/s$$

$$V_{avg} = \frac{1}{n} R_{avg}^{2/3} \times \sqrt{S_f}$$

$$\Rightarrow 0.385 = \frac{1}{0.03} \times \left(\frac{40 \times 3.5}{40 + 2 \times 3.5}\right)^{2/3} \times \sqrt{S}$$

$$S = 0.00003$$

$$\therefore L = \frac{4.5 - 2.51}{0.0000909 - 0.00003} = 32.67 \, km$$

A

BAPEX-2023
Non-36
Dept - 6X4 = 24

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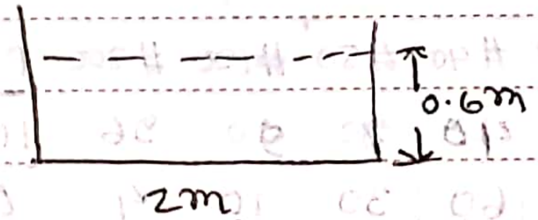
(1) Fill in the gaps:-

(a) Pitot tube measures the total head of fluid

(b) The depth of 2m wide channel is 0.6, the hydraulic depth 0.6m .



$* \; D = \dfrac{\text{Area}}{\text{Top width}}$

$= \dfrac{2 \times 0.6}{2}$

(c) Hydraulic depth does not depend on space (/distance in uniform flow.

(d) Specific ~~Potential~~ energy depends on hydraulic depth only.

* Pitot tube দিয়ে velocity নয়; pressure head ও measure করা হয়।

* Determine the field capacity of a soil for the following data –

Depth of root zone = 1.70 m, existing moisture = 8%, dry density of soil = 1450 kg/m^3, quantity of water applied to soil = 650 m^3, water loss due to deep percolation and evaporation = 10%, Area of irrigation land = 1000 m^2 [BWDB'22]

Soln: Water stored in root zone depth = $\frac{0.9 \times 650}{1000} = 0.585\,m$

water stored in root zone = $G_s \cdot d \cdot [F.C. - M.C.]$

$\Rightarrow 0.585 = \frac{1450}{1000} * 1.70\,[F.C. - 0.08]$

$\therefore$ F.C = 0.317 $\quad\therefore$ Field capacity = 31.73%

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* An irrigation field has following data –
Field capacity = 25%
Permanent wilting Point = 15%
~~Specific~~ unit weight = 14.03 kN/m^3
Root zone depth = 90 cm
Calculate the available moisture holding capacity of soil [Titas '21]

Solⁿ: Available moisture holding capacity

$$= 0.9 \times \frac{14.03}{9.81} \times [0.25 - 0.15]$$

$$= 0.1287 \text{ m} = 12.87 \text{ cm}$$ A

* The following data pertains to healthy growth of a crop:
→ Field capacity = 30%
→ Permanent wilting point = 11%
→ Effective depth of root zone = 200 mm
→ porosity of soil = 39%
→ Daily consumptive use of water = 12 mm

For healty growth moisture content must not fall bellow 25% of water holding capacity. Determine the watering interval in days.

Solⁿ:

$$\boxed{G_s = \frac{\text{porosity}}{\text{field capacity}} = \frac{0.39}{0.30} = 1.3}$$

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$\therefore$ Depth of water in root zone

$$= 1.3 * 700 * 0.75 * (0.30 - 0.11)$$

$$= 129.675\,mm$$

$\therefore$ watering interval $= \cancel{\dfrac{700}{129.675}} = \dfrac{129.675}{12} \approx 10.8\,days$

$\approx 10\,days$

11তম দিনে কিছু সময়ের জন্য পানির ঘাটতি থাকে,

* The field capacity & moisture content at the time of irrigation are 27% and 19%. The apparent specific gravity is 1.3 & the root zone depth is 100cm. Determine the time required to irrigate 2ha with a flow of 60 l/s, if the water application losses are taken to be 20% [BWDB'2016]

Soln: Net irrigation depth $= 1.3 \times 100 \times [0.27 - 0.19]$

$$= 10.4\,cm$$

$$= 0.104\,m$$

Field irrigation depth $= \dfrac{0.104}{0.8} = 0.13\,m$

Volume of irrigation water $= 0.13 \times 2 \times 10^4$

$$= 2600\,m^3$$

Time required $= \dfrac{2600}{60 \times 10^{-3} \times 3600\ m^3/hr}$

$$= 12.037\,hr \quad Ans$$

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Relation Between Duty & Delta

$$\Delta = \frac{864 B}{D}$$

Related math
Titas '21

Δ (delta) in cm
D = Duty (hectares/cumec)
B = Base period (days)

* What is the discharge capacity required at the outlet to irrigation 2600ha of sugercane having a kor depth of 17cm & a kor period of 30 days.

Soln: $\text{Duty} = \frac{864 B}{\Delta} = \frac{864 \times 30}{17}$
$= 1524.71$ ha/cumec

$\therefore$ Discharge capacity $= \frac{2600 \text{ ha}}{1524.71 \text{ ha/cumec}}$
$= 1.705$ cumec

* A canal commands an irrigation area of 400ha. The duty of water on the field during the peak period is 220ha/cumec. Determine the design discharge of the canal at the off take if the water loss in the canal is 30%? [BWDB'16]

Soln: Discharge $= \frac{400}{220} = 1.82$ cumec

design discharge $= \frac{1.82}{0.70} = 2.60 \text{ m}^3/\text{sec}$

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* A crop required 45cm of water during 120days of base period. The total rainfall during this period is 100mm. Assume the irrigation efficiency to be 60%. Calculate the area of land which can be irrigated with a canal flow of 0.01 cumec.

Soln: Duty of irrigation water $= \frac{864 \times B}{\Delta}$

$= \frac{864 \times 120}{(45-10)}$

$= 2962.286$ ha/cumec

effective canal flow $= 0.6 \times 0.01 = 0.006$ cumec.

$\therefore$ Area to be irrigated $= 2962.286 \times 0.006$

$= 17.773$ ha

Ans

* A pump is installed on a well to lift water and to irrigate rice crop, sown over 3 hectares of land. If duty for rice is 864 hectares/cumec on the field and pump efficiency is 48%, determine the minimum required input (HP) of the pump if the water level is 8m below the ground.

Soln: Discharge required $= \frac{3}{864} = \frac{1}{288}$ m³/sec

Input of the pump $= \frac{\gamma Q H}{\eta} = \frac{9.81 \times \frac{1}{288} \times 8}{0.48} = 0.567$ kW

$= 567$ W

$\frac{567}{746}$ HP

$= 0.76$ HP

Ans

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

Soln:

$$\Delta = \frac{140}{28} \times 7.5\,cm = 37.5\,cm \text{ Ans}$$

* The gross commanded area for a distributary is 6000 hectares, 80% of which is culturable. The intensity of irrigation for Kharif season is 25% & that for Robi season is 50%. If the average duty at the head of distributory is 700 hectares/cumec for Kharif ~~Rabi~~ season & 1700 for Rabi season, find the discharge required for design at the head distributory.

Soln:

Discharge required for Kharif season,

$$Q_K = \frac{0.8 \times 6000 \times 0.25 \text{ ha}}{700 \text{ ha/cumec}}$$

$$= 1.714\,m^3/sec$$

Discharge for Rabi season $= \dfrac{0.8 \times 6000 \times 0.50 \text{ ha}}{1700 \text{ ha/cumec}}$

$$= 1.41\,m^3/s$$

∴ Design discharge = maximum value

$$= 1.714\,m^3/sec$$

A

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* Determine the time required to irrigate a strip of land of 0.04 hectares in area from a tube-well with a discharge of 0.02 cumec. The infiltration capacity of the soil may be taken as 5cm/hr & the average depth of flow on the field as 10cm. Also determine the maximum area that can be irrigated from this tubewell.

[WRE MSc'21]

Soln:

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

$$t = \frac{10}{5} \ln\left(\frac{72}{72 - 0.05 \times 400}\right)$$

$= 0.65$ hr

$= 39$ min

$y = 10$cm

$f = 5$cm/hr $= 0.05$m/hr

$Q = 0.02\ m^3/sec$

$= 0.02 \times 60 \times 60$

$= 72\ m^3/hr$

$A = 0.04$ ha

$= 0.04 \times 10^4\ m^2$

$= 400\ m^2$

Max^m area that can be irrigated is $= \frac{Q}{f} = \frac{72}{0.05}\ m^2 = 1440\ m^2$

A

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* pH value for fertile soil 6.5 to 7.5*
[Titas'21]

* drinking water ~~ও~~, pH 6.5 to 8.5*

* Salinity of water is measured by electric conductivity values [BPDB'21]

EC values (micro mhos/cm)	Class
<250	Low salinity water (C_1)
250-750	medium salinity water (C_2)
750-2250	High salinity water (C_3)
>2250	very high salinity (C_4)

Sodium Absorption Ratio (SAR):

$$SAR = \frac{Na^+}{\sqrt{\frac{Ca^{2+} + Mg^{2+}}{2}}}$$

SAR value	Water type
0-10	Low sodium water (S_1)
10-18	Medium sodium water (S_2)
18-26 (Titas'21)	High sodium water (S_3)
>26	Very high sodium water (S_4)

*(a) What is the classification of irrigation water having the following characteristics: concentration of Na, Ca & Mg are 22, 3 & 1.5 milli-equivalents per litre respectively & the electric conductivity is 200 micro mhos/cm at 25°C.

(b) What problems might arise in using this water on fine textured soils?

(c) What remedies do you suggest to overcome this trouble?

Soln:

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

$10 < SAR < 18 \Rightarrow$ medium sodium water (S_2)

$EC = 200\,\mu$mhos/cm < 250

$\Rightarrow$ low salinity water (C_1)

Hence it is classified as $C_1 - S_2$ water

(b) Problems that might arise —

→ Soil becomes less permeable

→ It becomes plastic & sticky when wet

→ It stats crusting when dry

→ Its pH increases

(c) Gypsum ($CaSO_4$) addition in soil or water to overcome sodium hazards posed by the given water.

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Duty: It is defined as the area of land in hectares which can be irrigated for growing any crops if one cumec of water is supplied continuously to the land for the entire base period of crop.

Titas'14

Unit - hectares/cumec

Delta: The total depth of water over irrigation field required by a crop to come to maturity is called delta.

Gravity Dam: A gravity dam is such a structure which is is designed in such a way that it's own weight resists the external forces.

Forces acting on gravity dam [PGCB'17]

→ Water pressure
→ Uplift pressure
→ pressure due to earthquake force
→ Silt pressure
→ Wave pressure
→ The stabilizing force (self weight of dam)

Irrigation Channel Design:

Manning's coefficient, $n = \dfrac{d_{50}^{1/6}}{21.10}$

or $n = \dfrac{d_{75}^{1/6}}{24}$

Max^m hydraulic radius, $R_{max} = \dfrac{d}{115}$

যখন θ, φ দেওয়া থাকবে $R_{max} = \dfrac{kd}{0.75 \times 115}$

যেখানে, $k = \dfrac{\tau'_c}{\tau_c} = \sqrt{1 - \dfrac{\sin^2\theta}{\sin^2\phi}}$

* Irrigation canal এর ক্ষেত্রে, $R = y$ (depth)

* Critical shear stress, $\tau_c = \tau^* (G_s - 1)\gamma_w D$

$\Rightarrow$ τ^* = dimensionless effective stress

again, $\tau_c = \gamma_w R S$

discharge, $Q = \dfrac{1}{n} A R^{2/3} S^{1/2}$

$Q = \dfrac{1.486}{n} A R^{2/3} S^{1/2}$ [ft এককে দেওয়া থাকলে]

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D কে অবশ্যই meter এ নিয়ে n, R বের করতে হবে

* An irrigation channel is to be constructed in a coarse alluvium gravel with D_{75} size of 5cm. The channel has to carry 3 cumecs of discharge and the longitudinal slope is 0.01. Find the minimum width of the channel.

Soln:

$$R_{max} = \frac{d}{11S} = \frac{\frac{5}{100}\,m}{11 \times 0.01} = 0.455m$$

$$n = \frac{d_{75}^{1/6}}{24} = \frac{(0.05)^{1/6}}{24} = 0.0253$$

$$Q = \frac{1}{n} A R_{max}^{2/3} S^{1/2} = \frac{1}{n} (b_{min} \times R_{max}) \times R_{max}^{2/3} S^{1/2}$$

$$\Rightarrow 3 = \frac{1}{0.0253} \times b_{min} \times (0.455)^{1+\frac{2}{3}} \times \sqrt{0.01}$$

$$\Rightarrow b_{min} = 2.82m$$ Ans

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* A canal is to be designed to carry a discharge of 56 cumec. The slope of the canal is 1 in 1000. The soil grain size is 5cm. Assuming the canal to be unlined & of a trapezoidal section, determine the depth of the canal. Take $\phi = 37^\circ$

Solⁿ:

let $\theta = 30^\circ$

Now, $\frac{\tau_c'}{\tau_c} = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1 - \frac{\sin^2 30^\circ}{\sin^2 37^\circ}}$

$= 0.557$

$\therefore R_{max} = \frac{0.552 \times \frac{5}{100}}{0.75 \times 11 \times \frac{1}{1000}} = 3.375m$

$\therefore$ depth $= 3.375m$ A

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* The value of dimensionless effective stress is ~~0.045~~ 0.045. Find the value of incipient depth and effective velocity of a wide river channel with the bed sediment size of 2mm & bed slope of 0.0001 [BWDB'16]

Solⁿ:

$$\tau_c = \tau^* (G_s - 1)\gamma_w d$$

$$= 0.045(2.65-1)\times 9.81\times \frac{2}{1000}$$

$$= 1.457\times10^{-3}\ KN/m^2$$

Take Sp.Gr = 2.65

again, $\tau_c = \gamma_w RS$

$$\therefore R = \frac{\tau_c}{\gamma_w S} = \frac{1.457\times10^{-3}}{9.81\times0.0001}$$

$$= 1.485m$$

$\therefore$ depth, $y = 1.485m$ An

$$n = \frac{d_{50}^{1/6}}{21.10} = \frac{(2\times10^{-3})^{1/6}}{21.10} = 0.0168$$

$$\therefore V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$= \frac{1}{0.0168}\times(1.485)^{2/3}\times\sqrt{0.0001}$$

$$= 0.774\ m/s$$ A

Regime Channel Design:

যে channel এর flow এমন থাকে যে তাতে silting বা scouring হয়না তাকে Regime channel বলে।

True Regime:

(1) Discharge is constant
(2) Flow is uniform
(3) Amount of silt is constant
~~(4) Silt is constant grade~~
(4) Silt grade is constant
(5) channel is flowing through a material which can be scoured as easily as it can be deposited.

Design of Regime Channel According to Lacey's Theory –

⇒ silt factor, $f = 1.76\sqrt{d_{mm}}$ [d অবশ্যই mm এককে হবে।]

⇒ velocity, $V = \left[\frac{Qf^2}{140}\right]^{1/6}$ m/s

⇒ Hydraulic mean depth, $R = \frac{5}{2}\left(\frac{V^2}{f}\right)$ = Regime scour depth

⇒ bed slope, $S = \frac{f^{5/3}}{3340\,Q^{1/6}}$

⇒ Perimeter, $P = 4.75\sqrt{Q}$ = Regime width

*** ⇒ Normal scour depth $= 1.35\left(\frac{q^2}{f}\right)^{1/3}$

Date : / /

Sun | Mon | Tue | Wed | Thu | Fri | Sat

* Calculate the scour depth of a river having flow $2500 m^3/s$, bed material size is 0.15mm [BWDB'22]

~~Soln~~

$$\cancel{f = 1.76\sqrt{0.15} = 0.68}$$

~~Normal scour depth~~, $\cancel{R = 1.35\left(\frac{Q^2}{f}\right)^{1/3}}$

$$\cancel{= 1.35\left(\frac{2500^2}{0.68}\right)^{1/3}}$$

Soln: $f = 1.76\sqrt{0.15} = 0.68$

$$v = \left[\frac{Qf^2}{140}\right]^{1/6} = \left[\frac{2500 \times (0.68)^2}{140}\right]^{1/6}$$

$$= 1.42 m/s$$

$$\therefore \text{Scour depth} = \frac{5}{2}\left(\frac{v^2}{f}\right)$$

$$= \frac{5}{2} \times \frac{(1.42)^2}{0.68}$$

$$= 7.43m$$

যেহেতু q এর বদলে Q দেওয়া তাই এখানে Regime scour depth ই বের করতে হবে।

Date :
Sun Mon Tue Wed Thu Fri Sat

* Calculate scour depth for a river having width of 500m & unit discharge $50 m^3/s/m$. ~~d_{50} = 0.1mm~~ $d_{50} = 0.1mm$. Design discharge is 80% of bank full discharge. [BWDB'20]

⇒ Soln:

$f = 1.76\sqrt{0.1} = 0.537$

Normal scour depth $= 1.35\left[\frac{q^2}{f}\right]^{1/3}$

$= 1.35\left[\frac{(0.8\times50)^2}{0.537}\right]^{1/3}$

$= 19.42m$ A

* Types of scour occurs near bridge piers → (Padma bridge Project'2013)

→ Aggradation & degradation scour

→ contraction scour

→ Local scour

Design an irrigation channel in alluvial soil according to Lacey's silt theory. Given the following data — [PGCB'20] [DPDC'20] [SGCL'20]

discharge $= 15\ m^3/sec$

Lacey's silt factor $= 1.0$

Channel's side slope $= \frac{1}{2}H : 1V$

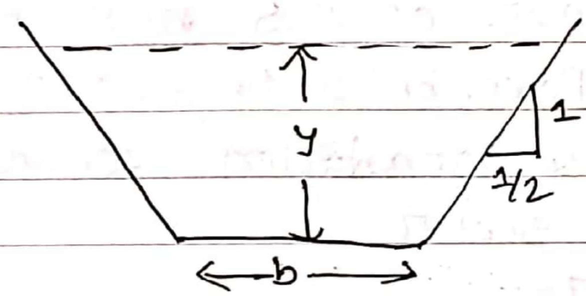
Soln:

(1) velocity, $v = \left[\frac{Q \times f^2}{140}\right]^{1/6} = \left[\frac{15 \times 1^2}{140}\right]^{1/6} = 0.69\ m/s$

(2) Area, $A = \frac{15}{0.69} = 21.74\ m^2$

(3) perimeter, $P = 4.75\sqrt{Q} = 4.75\sqrt{15} = 18.4\ m$

(4)



$A = (b + sy)y$

$= (b + \frac{1}{2}y)y$

$= by + \frac{y^2}{2}$

$\therefore by + \frac{y^2}{2} = 21.74$ — (i)

$P = b + 2y\sqrt{s^2+1}$

$\Rightarrow 18.4 = b + 2y\sqrt{\frac{1}{4}+1}$

$\Rightarrow b + \sqrt{5}y = 18.4$

$\Rightarrow b = 18.4 - \sqrt{5}y$ — (ii)

from eqn (i) & (ii)

$(18.4 - \sqrt{5}y)y + \frac{y^2}{2} = 21.74$

$\Rightarrow 18.4y - \sqrt{5}y^2 + \frac{y^2}{2} = 21.74$

Solving this, $y = 1.35\ m$

$\therefore b = 18.4 - \sqrt{5} \times 1.35 = 15.4\ m$

(5) Hydraulic radius, $R = \frac{5v^2}{2f} = \frac{5 \times 0.69^2}{2 \times 1}$

$= 1.2\ m$

(6) Bed slope, $S = \frac{f^{5/3}}{3340\, Q^{1/6}} = \frac{1^{5/3}}{3340 \times 15^{1/6}}$

$= 0.00019$

A

River Training:

River training covers all those engineering works which are constructed on a river so as to guide and confine the flow to the river channel and to control and regulate the river bed configuration, thus ensuring safe and effective disposal of floods and sediment loads. stabilising and training the river along a certain allignment with a suitable waterway is therefore the first and formost aim of river training.

Objectives of River Training:

(i) Provide a safe passage to flood discharge without overflowing to the bank for protection of cultivated or inhabitated area.

(ii) Prevent ~~outflaking~~ outflanking ([illegible]) of structures like a bridge, weir or aqueduct constructed accross the river

(3) Protect the bank from errosion and improve the alignment by stabilizing the river channel.

(4) Deflect the river away from the bank which it might be attacking.

(5) Provide minimum depth of flow and a good course for navigation purpose.

(6) Ensure effective disposal of sediment load.

Classification of River Training: (Important)

(i) <u>High Water Training</u>

⇒ Primary purpose is flood control

⇒ It aims at providing sufficient river cross-section for the safe passage of maximum flood. It is also concerned with making adjoining area flood proof by construction of ~~dykes~~ levees.

(ii) <u>Low Water Training:</u>

⇒ Primary purpose to provide sufficient water depth for navigation during lower water period

⇒ Accomplished by concentrating and enhancing the flow in the desired channel by closing other channels.

(iii) Mean Water Training:

⇒ aims at efficient disposal of suspended load and bed load and thus to preserve the channel in good shape

⇒ The mean water training is the most important type and forms the basis on which the former two are planned.

Important

Methods For River Training: (OPSC)

(i) Marginal Embankments/levees
(ii) Guide banks
(iii) Groynes or spurs
(iv) Artifical cut-offs
(v) Pitching of banks & provision of launching aprons.
(vi) Pitched islands
(vii) Sills, Bandalling etc

Guide Bank:

মূলকাজ ২ টা

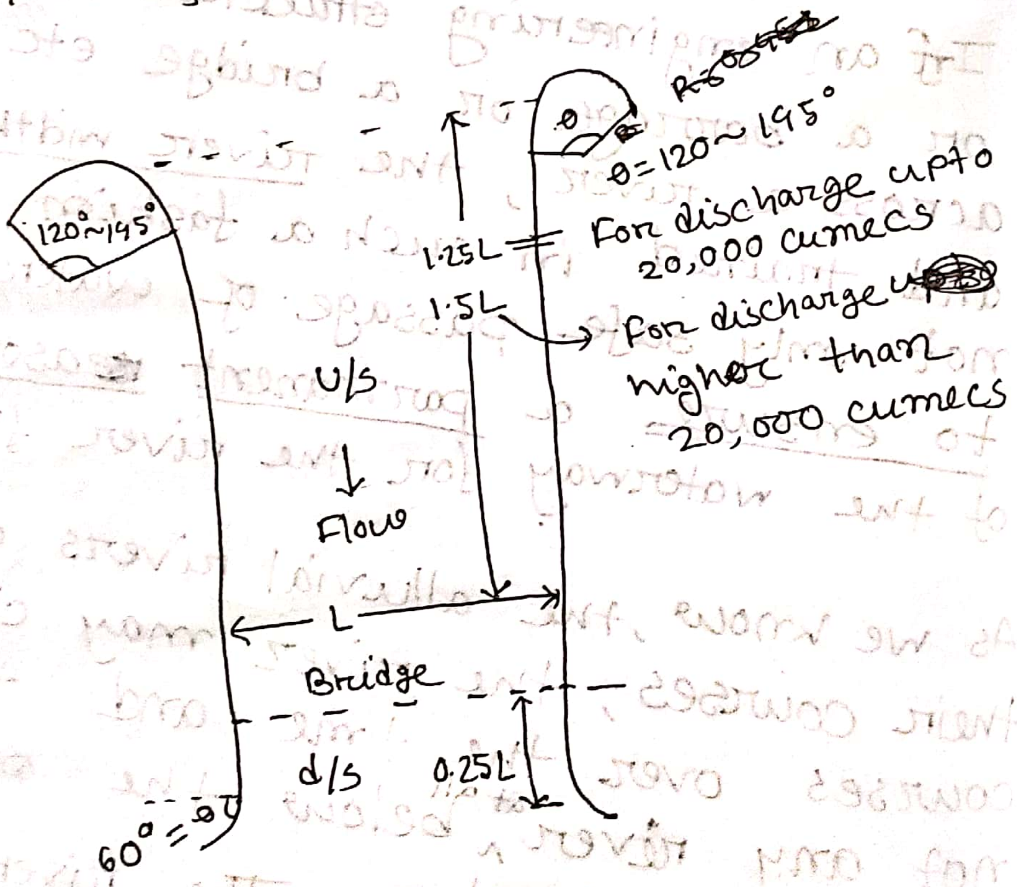
১/ নদী যাতে গতিপথ পরিবর্তন করে অন্য দিক দিয়ে প্রবাহিত হতে না পারে। কারণ তাহলে bridge বা অন্য যে structure বানাবো সেটা useless হয়ে যাবে।

২/ নদীর width কমানো। পুরো নদীর প্রস্থ জুড়ে ব্রীজ/weir/dam বানানোর ~~খুঁটি~~ খুঁটিনাটির জায়গা না। প্রচুর খরচ হবে।

If an engineering structure, such as a weir or a barrage or a bridge etc is constructed across a river, the river width is reduced and trained in such a fashion as to ensure not only safe passage of water but also to ensure a parmanent reasonable width of the waterway for the river flow.

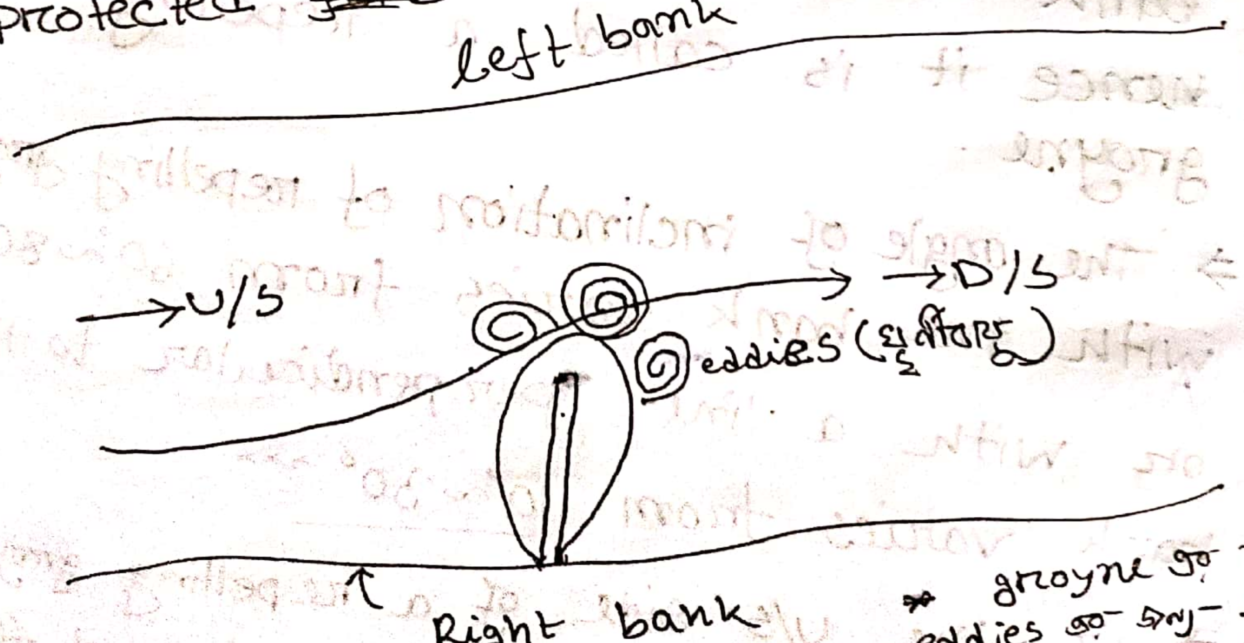
As we know, the alluvial rivers do shift their courses, the river may change it courses over the time. and there may not any river at all below the presently constructed bridge. The river may be found to be flowing away from it,

necessitating the construction of another structure.
Besides it is unwise and uneconomical to span the entire width of the river and to expose structure for deep scour. Hence, a structure such as a weir / bridge / barrage etc is extended in a smaller width of the river.



Spurs or Groynes :

Groynes are the embankment type structures, constructed (আড়াআড়ি) transverse to the river flow, extending from bank into the river. That is why, they may also called "Transverse (আইল) Dykes". They are constructed in order to protect the bank from which they are extended, by deflecting the current away from the bank. As the water is unable to take a sharp embayment (হঠাৎ মোড় ঘুরতে অক্ষম); the bank gets protected. ~~for~~



* groyne এর nose এ eddies এর জন্য প্রচুর চাপ থাকে পানির, তাই ওই অংশ heavily pitched থাকে তাই বেশি হয়, নিচেও অংশ চিকন।

The nose of groyne is subjected to tremendous action of water and has to be heavily protected by pitching etc.
The action of eddies reduces from the head towards the bank, therefore the thickness of slope pitching and aproan can be reduced accordingly.

Repelling Groynes:

⇒ A groyne pointing u/s tends to repel the river flow away from the bank on which it is provided and hence it is called a repelling groyne.

⇒ The angle of inclination of repelling groynes with the bank varies from $60° \sim 80°$ or with a line perpendicular to the bank varies from $10° \sim 30°$ ***

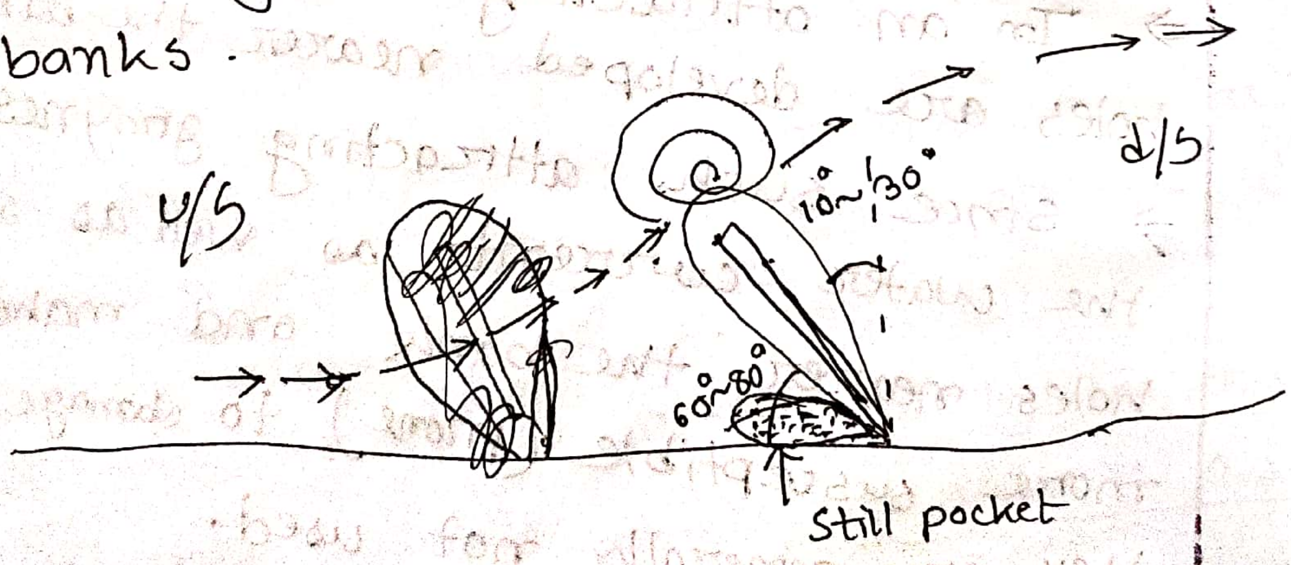
⇒ On the u/s side of a repelling groyne, a still water pocket is formed and the suspended sediment carried by the river gets deposited in the pocket.

⇒ The head of a repelling groyne cause disturbances in the flow at its nose.

⇒ Heavy scour occurs at the nose due to eddy formation.

⇒ The head of a repelling groyne needs a very strong protection because it is subjected to direct attack of a swirling current.

*** normal groyne (perpendicular alignment) is generally used on convex banks, repelling groynes is used on concave banks.



Repelling groyne

*** When the length of an ~~u/s~~ repelling groyne is small such that it changes only the direction of flow without repelling it it is called a <u>deflecting groyne</u>

<u>Attracting groyne:</u>

⇒ A groyne pointing d/s tends to attract the river flow towards the bank on which it is provided. Hence it is called an attracting groyne.

⇒ The angle of inclination of an attracting groyne with the bank may be in the range of ~~30° to 60~~ 60° ~ 80°

⇒ In an attracting ~~hole~~ groynes, scour holes are developed nearer the bank.

⇒ Since such attracting groynes bring the water current as well as scour holes nearer the bank and make it more susceptible (prone) to damage, they are generally not used.

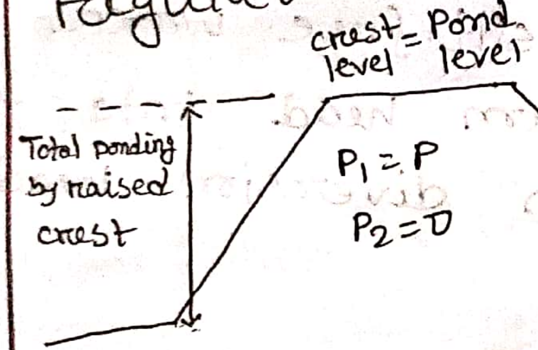
Diversion Head Works

An irrigation canal takes its supply from a river or a stream. In order to divert water from the river into the canal it is necessary to construct certain works or structures across the river in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of silt-free water with a certain minimum head into the canal ~~are~~ known as diversion head works.

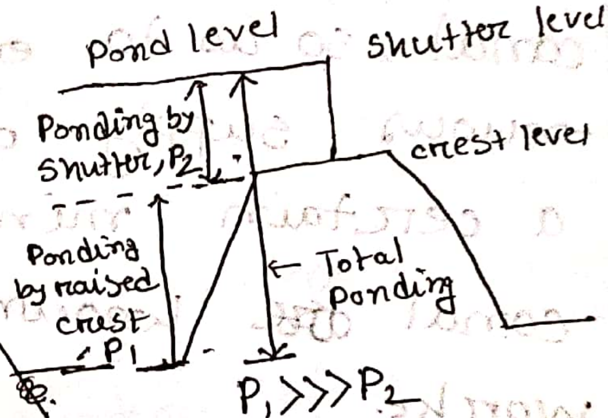
Weir & Barrage:
To maintain minimum head, water level raising or ponding in the u/s side of obstruction structure is required.

If the major part or the entire ponding of water is achieved by a raised crest and a smaller part or nil part of it is acheived by the shutters then ~~this~~ this barrier is known as a weir.

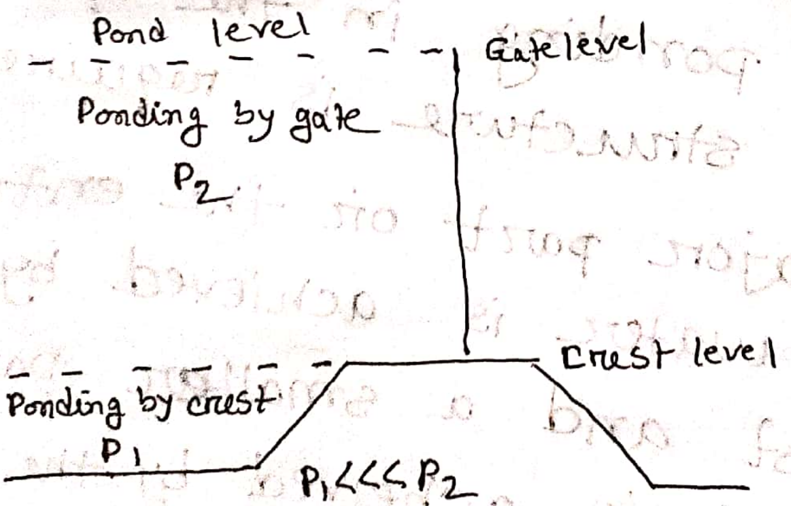
On the other hand, if most of the pond is done by gates/shutter and a smaller or nill part of it is done by the raised crest, then the barrier is known as a barrage or a river regulator.



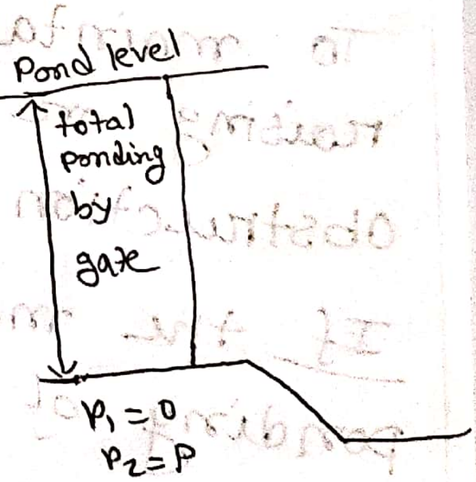
(a) weir without shutter



(b) weir with shutter



(a) Barrage with small raised crest



(b) Barrage without raised crest

If most of the ponding or entire ponding is done by a parmanent raised crest as in a weir, then the afflux caused during high flood is quite high.

on the other hand, if most of the ponding is done by gates as in a barrage, then the gates can be opened during high floods and the afflux will be nill or minimum.

Hence, barrage gives less afflux and better control upon river flow.

A weir is relatively cheaper but will lack the effective control over river flow as compared to barrage.

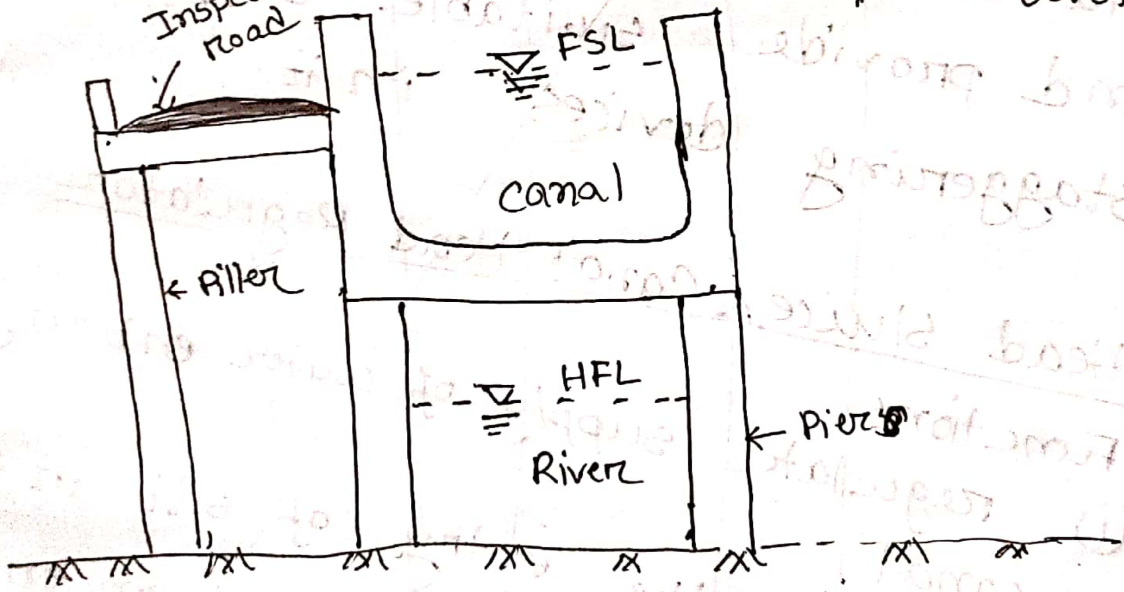
But a barrage can be provided with a roadway accross the river at a small additional cost. (barrage cum bridge)

Aqueduct:

The hydraulic structure in which the irrigation canal is taken over the drainage (such as river, stream) is known as aquiduct.

This structure is suitable when bed level of canal is above the highest flood level (HFL) of drainage (river).

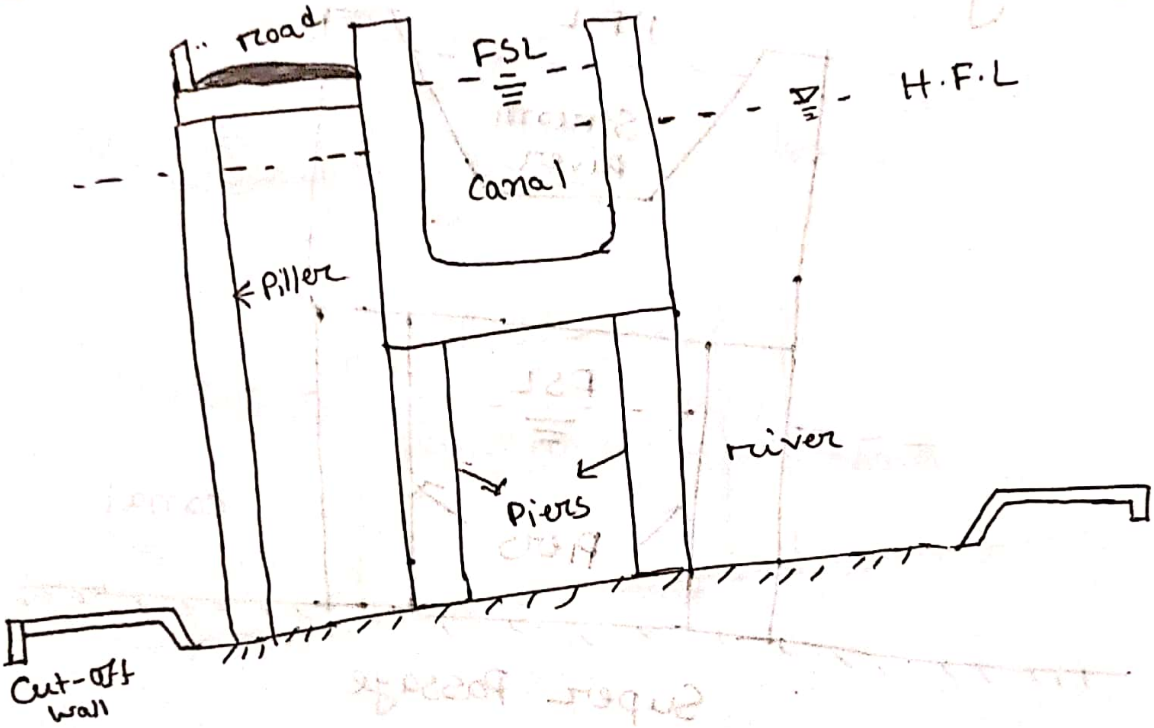
* FSL = Full Supply Level



Aqueduct

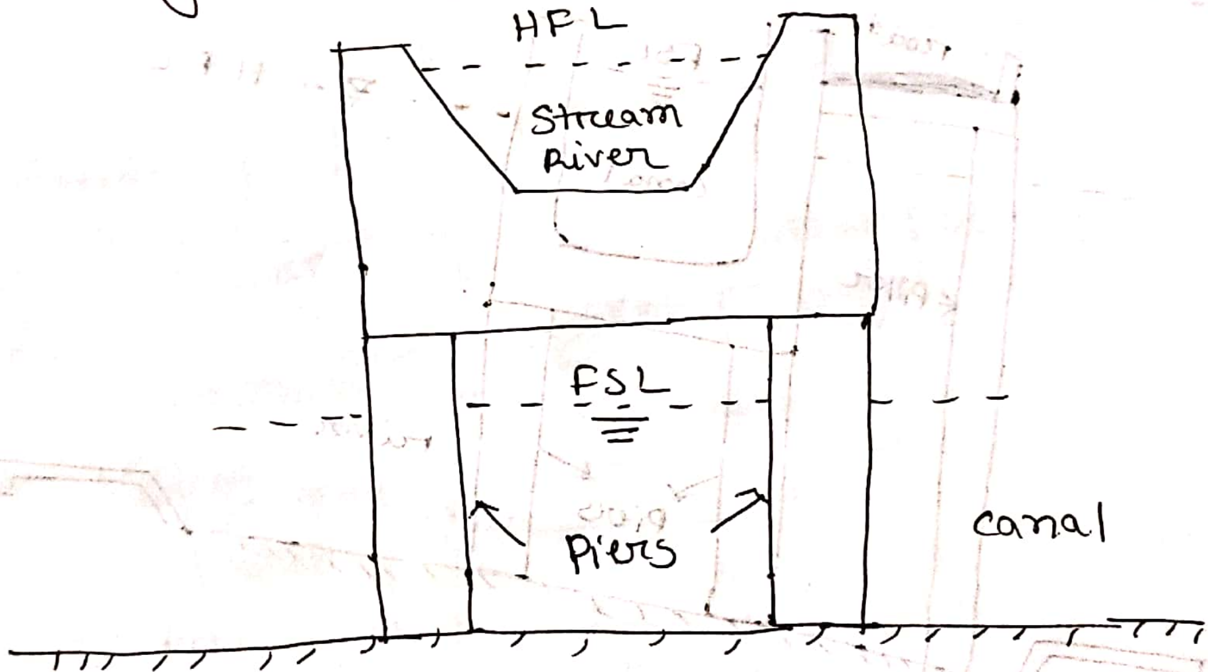
Siphon Aqueduct : In a hydraulic structure where the canal is taken over the drainage but the drainage water cannot pass clearly below the canal. It flows under siphonic action.

This structure is suitable when the bed level of canal is below the highest flood level.



Super Passage: The hydraulic structure in which the drainage (river) is taken over the irrigation canal is known as super passage.

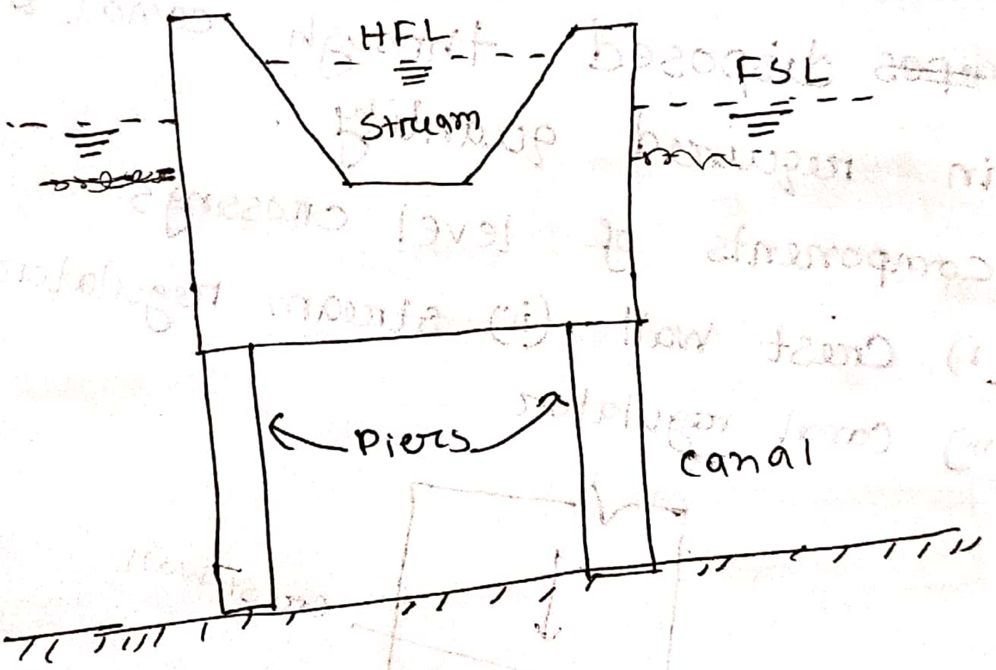
Bed level of drainage is above the FSL of canal. The water of the canal passes clearly below the drainage.



Super Passage

Siphon Super Passage: The hydraulic structure in which the drainage is taken over the irrigation canal, but the canal water passes below the drainage under siphonic action is known as ~~sp~~ siphon super passage.

This structure is suitable when the bed level of drainage is below the full supply level of the canal.



Level Crossings: When the bed level of canal and the stream are approximately the same and quality of water in canal & stream is almost same, the ~~cross~~ level crossing is constructed where water of canal & stream is allowed to mix with the help of regulators. Water is ~~dipos~~ disposed through canal & stream in required quantity.

components of level crossings

(i) Crest wall, (ii) stream regulator

(iii) Canal regulator.